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8. Papers should average about 12,000 words in length and should be no longer than 18,000 words. As an approximation, each full page of typed text, table, or illustration is the equivalent of 300 words.

Further information concerning the preparation of technical papers is contained in the "Technical Publications Handbook" which can be obtained from the Society.

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ERRATUM

IRRAWADDY RIVER SYSTEM OF BURMA

by John B. Alexander and Henry R. Norman, Members, ASCE

The authorship of the Paper "Irrawaddy River System of Burma," which was published in the September 1958 issue of the Journal of Waterways and Harbors Division and as Proceedings Paper No. 1776, was erroneously stated in those publications.

At the request of the Division, the Paper was prepared jointly by Mr. John B. Alexander who was, at the time the Paper was prepared, Deputy General Manager and Chief Planning Engineer for Tippetts-Abbett-McCarthy-Stratton (formerly Knappen-Tippetts-Abbett Engineering Co.) in their Rangoon, Burma, Office, and Mr. Henry R. Norman, at that time Transportation Engineer, Ports and Waterways, of Tippetts-Abbett-McCarthy-Stratton, also in their Rangoon office. The paper was based on studies of the Irrawaddy River prepared as part of comprehensive engineering and economic surveys and reports by Tippetts-Abbett-McCarthy-Stratton for the Government of the Union of Burma.

Unfortunately, in both the September issue of the Journal and in Paper No. 1776 (reprinted from the Journal), Mr. Alexander's name and affiliation were omitted and sole authorship credit was given to Mr. Norman, citing at the same time Mr. Norman's present affiliation rather than his affiliation when Mr. Alexander and he prepared this paper.

Robert W. Abbett, M. ASCE

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VERTICAL-LIFT GATE DESIGN FOR ICE HARBOR LOCK[a]

Howard M. Rigler[1] and Edmund H. Chun,[2] A. M. ASCE
(Proc. Paper 1873)

SYNOPSIS

This paper discusses the details of the design of the downstream lift gate for the Ice Harbor Lock, the justification for its selection and some of the problems encountered. Also presented in a more general way are the major features of the lock as a whole.

INTRODUCTION

Ice Harbor Lock and Dam is located on the Snake River 9.7 miles upstream from its confluence with the Columbia River. The single-lift navigation lock is 86 feet wide, 675 feet long and has a maximum life of 102 feet. This is 10 feet greater than the McNary lock and will be, when completed, the highest single lift lock in the world.

Planning studies for the Ice Harbor Lock included the design for a conventional arched girder miter gate, 120 feet high, each leaf weighing 460 tons. Later studies in the design stage led to the lowering of the downstream gate sill 4 feet. This increased the height of the gate by a similar amount and increased the weight of each leaf from 460 to 475 tons, or a total of 950 tons for the entire gate.

Miter gates, which have been used for many years, have given excellent, trouble free service. However, as the height of these gates has increased so have operational difficulties increased. At the McNary Lock there is a pronounced chattering as the gate is being closed and approaches the mitering

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a. Presented at the ASCE Convention in Portland, Oregon June, 1958.

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2. Asst. Chief, Structural Design Section, U. S. Army Engineer District, Walla Walla, Wash.

position. The exact cause of this is not known but considering that the gate is 105 feet in height with one operating arm attached to the top of the gate and the lower 15 to 20 feet of the gate submerged, it is not surprising. In order to remedy this chattering and vibration, the diagonal members, which resist distortion of the gate as it hangs free on the gudgeon linkage when open and also resist torsion in the entire leaf when it is in motion, should be much stiffer and heavier than designed. The Ice Harbor miter gate, 125 feet in height and with a minimum submergence of 21 feet and a 20-foot longer lever arm in the couple between the center of resistance and the center of effort would require a large additional amount of metal, particularly in the diagonal members to effect the stiffness necessary to resist torsion stresses adequately.

A lift gate with an across-locks concrete girder appeared to offer definite advantages. Preliminary design studies were initiated for the purpose of comparing both construction costs and facility of operation.

The Lift Gate

Lift gates are not new. The Prinz Bernhard Sluis at Tiel, Netherlands has a downstream lift gate. The Donzere-Mondragon Lock on the Rhone has a downstream lift gate with an across lock curved girder which acts as a dam above the gate when the gate is in fully closed position. Overhead clearance for navigation has been established at 50 feet on the Snake River. Tailwater elevations at Ice Harbor vary from 340 to 365 so that an across-locks girder or dam with its soffit at Elevation 410 gives clearance for all river stages except the highest floods of record which are of rare occurrence. With the downstream lock sill at Elevation 321 the gate need be only 91 feet in height. Normal upper pool elevation is 440 so that the top of the gate will be submerged 29 feet with the lock chamber full.

Preliminary calculations indicated that the most economical design for the gate girders would be tied arches so spaced that each girder would support an equal load. Since each girder carries an equal load, they are identical in design and will deflect uniformly under load. Some economy in fabrication costs will result from this uniformity of design. There are 18 tied arches with a minimum spacing of 3 feet 5 inches and maximum spacing of 9 feet 5 inches. Since the top of the gate is submerged when the lock is full, the top member must be a plate girder with a curved compression flange. The arches are 87 feet 5 inches long, have a radius of 80 feet 0 inches and a middle ordinate of 13 feet and an all over width of 15 feet.

Each arch consists of a curved compression member—an ST 18 WF 150 with skin plate flange—total area 92.8 sq. in. and normal working stress of 19,250 psi. The straight tension member is a 33WF 240, area 70.52 sq. in., normal working stress 21,500 psi. The skin plate is 1-3/16 inches thick over the entire surface of the gate. The uniform thickness was chosen in order to make all arches of equal resisting moment. Vertical intercostals are ST sections. Since no unbalanced load is possible on the gate, a system of web members was not considered necessary. ASTM-A242, low alloy steel will be used throughout the gate.

An interesting sidelight on the detailed design of the gate for optimum arch design and spacing, skin plate thickness, and intercostal sizes is the fact that all such calculations were made on the IBM-650 digital computer which is

installed in the office of North Pacific Division, Corps of Engineers in Portland, Oregon. A total of ten gates were programmed, the initial difference being the arch spacing at the bottom of the gate. The minimum spacing was, of course, governed by working space requirements. The choice of either ASTM-A242 and A7 steel was also incorporated in the program. There were approximately 450 commands in the program and in addition, a very considerable amount of information from the steel handbook had to be impressed on the memory drum. The program worked well. It is the opinion of the writer that problems of this sort, with many variables and in which a great many similar, repetitive and tedious computations must be performed are well adapted to digital computer processing. The cost of the programming and machine time was far below the estimated design cost, had the work been done with slide rule and pencil.

The lift gate which weighs 1,400,000 pounds is balanced by two counterweights, 10 feet by 10 feet by 36 feet, consisting of a steel shell of 1/2-inch plate, reinforced with ST sections and filled to within 3 feet of the top with 200# pcf concrete. Final balancing will be accomplished with cast-iron pigs which will average 75 to 100 pounds each. Twelve (12) 2-1/4-inch diameter stainless steel cables which pass over the 18-foot diameter operating sheave will connect the gate and counterweights. As is customary in lift bridge design, these cables are designed for a large safety factor—6 when the gate is in motion and approximately 9 when at rest. The cables have a pin connection at the gate and an adjustable connection at the counterweight end. Replacement of these cables can be made at any time with the gate and counterweight in such position that the cables are balanced over the sheave.

The gate bears, when under full load, by means of two adjustable shod bearing plates on an embedded bearing plate in the gate slot. The cables are so positioned that the center of lifting effort is slightly upstream of the center of gravity of the gate. When lifting power is applied the gate tilts upstream thus clearing the bearing surfaces and side and top seals. The lower wheels are in a 5/8-inch depression in the bearing plate when the gate is in full down position and under load, but as the gate rises, the wheels come out of the depressions and clear the side seals and the bearing plate from any possibility of drag on their seating surfaces. The size of the gate slots permits the top wheel to bear on the upstream seating plate when the top of the gate has tilted about one inch upstream.

One feature (or advantage) in lock operation with the downstream lift gate and the upstream tainter gate is that there is practically no possibility of simultaneous failure of both gates. The interlocking controls will normally not permit both gates to be open at the same time. However, should the interlocks fail, and manual operation resorted to, either gate, or both, can be closed against any flow that would result from both gates being open. A consideration of these facts has altered the usual system of furnishing emergency stoplogs, both upstream and downstream. In the case of Ice Harbor, normal maintenance of the upstream gate will be periodically necessary. For the purpose of the upstream sill unwatering a floating caisson will be furnished.

This caisson is patterned after those which are used to close the drydocks at Bremerton Navy Yard and which have been operated successfully since 1917. No stoplogs or caisson will be supplied for emergency downstream closure. Slots in the downstream monoliths will provide for the use of either. The upstream caisson is of sufficient height to operate satisfactorily at normal tailwater elevation and in the event that closure is required both

upstream and downstream, the downstream stoplogs at McNary Lock can be borrowed, transported by derrick barge, positioned by this barge and removed and returned when they are no longer required.

The across-locks girder is a reinforced concrete box girder 38 feet in depth and 25 feet in width. The walls of the box are 3 feet thick and there are 8 transverse diaphragms 2 feet thick. The horizontal seal plate for the gate top seal is embedded in the lower part of the girder. The girder also serves as an access bridge, a dam for the top 30 feet and as a tunnel for utilities.

The towers which house the lifting machinery and also serve as the upper part of the counterweight shaft are reinforced concrete. The machinery floor is at Elevation 510, 62 feet above the lock deck. Vertical load 2000K, 21,600 foot/K torque from lifting machinery and 33,500 foot/K moment due to eccentric loading.

Operating Machinery

The gate is operated by two electrically driven hoists which are located at the 510 elevation deck in the towers at each side of the lock. The two counterweights are each attached to the gate by means of twelve 2-1/4 inch diameter stainless steel ropes, each set of twelve ropes passes over an 18-foot diameter driving sheave and the ropes are dead ended at the gate and at the counterweight. Means for adjusting and equalizing the tension on each rope is provided at the counterweight connections. The driving sheave of each hoist is driven by an attached ring gear through a pinion gear, gear reducer, and motor. A brake is provided to stop the gate motion at any position. The hoists are designed to raise or lower the gate through a total vertical travel of 89 feet in approximately four minutes. Design acceleration and deceleration are uniform, 0.01 foot per second, squared, and maximum design velocity is 0.5 foot per second.

The gate will normally be operated under approximately balanced head conditions, with the lock and tailwater surfaces at about the same elevation; however, the hoists are designed to operate the gate starting from the closed position, with about 3 feet unbalanced head on the upstream side of the gate and with maximum tailwater on the downstream side. The 75 HP motor is sufficient to operate the gate under the unbalanced head at maximum torque with a combined machine and rope efficiency of 55 per cent and yet is not likely to cause slippage between sheave and ropes at maximum torque with an overall efficiency of 85 per cent. The maximum velocity and maximum effort required near the mid-point of the opening cycle were used to determine the minimum horsepower requirements. Conservative friction coefficients have been used in the design. A coefficient of 0.5 is assumed for metal to metal sliding between gate and seats and 0.085 for sliding between ropes and the driving sheave.

A triple reduction speed reducer of helical or herringbone type is provided to reduce the motor speed to that required at the pinion gear. Ratio of the reducer is 176:1. The ratio between the pinion and the sheave ring gear is 11.8:1 and the total reduction, at a motor speed of 1100 RPM is approximately 2070.

The pitch diameter of the driving sheave is 216 inches (18 feet). This large size is required because of the proportions and weight of the structures. Grooves are provided in the sheave for each of the twelve 2-1/4 inch diameter

wire ropes and the ropes contact the sheave on one-half of the circumference. The brake is electrically operated. It will normally serve to stop the gate at or near the raised and lowered positions but has sufficient capacity to stop the gate promptly when the gate is traveling at maximum velocity.

Component parts of the machinery are designed for a factor of safety of five based on normal loading and, in addition, each part is designed for a unit stress not to exceed 75 per cent of the yield point of the materials at loads resulting from the maximum torque of the drive motor. The maximum torque is assumed to be 280 per cent of the normal torque.

An air bubbler system will be provided to clear trash from and prevent ice formation in the gate slots, to prevent accumulation of trash on top of the gate when emptying the lock, and to prevent formation of ice on the gate at tailwater level. The top and side seal plates will be heated to prevent the formation of ice. The heating medium will be electrically heated oil which will be circulated through embedded tubing.

Operation and Controls

The gate may be operated normally only when the upstream tainter gate and the filling valves are closed and the emptying valves are open. This is accomplished by means of an interlock system, which also protects other components of the lock from being operated out of sequence. This interlock may be bypassed for maintenance or other unusual purposes.

Automatic leveling of the gate while it is being raised or lowered will be applied so that the maximum vertical skew will not exceed one inch. The machinery will be shut down automatically should this skew exceed one inch. The automatic leveling control mechanism provides the necessary error signal to a discriminating circuit which controls the speed of the river side drive motor through a rotating amplifier to maintain the gate in a level condition.

There will be three control stations for either full or partial control of the gate. For normal operation the gate will be operated from the control desk in the control house on the river side of the lock. Provisions will also be made in each tower for machinery operation for maintenance or emergency purposes.

In opening the gate, as it reaches the raised position the control circuit is opened by a contact in the limit switch driven by the hoist machinery. Back up protection is provided by a hatch type limit switch operated by the gate itself. In lowering the gate, as it approaches the lower limit, the speed will be reduced and the gate stopped approximately 6 inches above the bottom seal plate. The gate will then be jogged down at the slow speed to the seated position. A vernier scale on the gate position indicator will be provided to aid in this operation.

Maintenance

Ordinary maintenance and repair to the counterweight and access to the gate wheels and end bearing plates is made possible by means of an 18 by 32 foot room at Elevation 413. The counterweight shaft is in the center of this room. A watertight bulkhead through the end wheel guide slot provides access for removal and maintenance of the gate wheels. A movable platform consisting of two heavy girders mounted on wheels is located at Elevation 428. This

is necessary during initial construction as a support for the counterweight and may be used in maintenance to support the counterweight independently of the gate. For ordinary maintenance of the gate the lock need not be unwatered. The gate is merely hoisted above water level and any necessary work can then be done.

CONCLUSION

The final choice of this lift gate was based on cost. Two post derricks, one set of downstream stoplogs, a retractible or bascule type bridge crossing the locks near the upstream sill were all made unnecessary by its use. The lock chamber was shortened 45 feet with consequent saving in first cost and also operation costs because of faster filling and emptying time. The clear length of 675 feet is still maintained, the difference, of course, being the space required by the swing of the miter gate leaves.

While final costs have not been definitely ascertained, careful estimates have shown a saving of over $700,000 when compared to the cost of the conventional miter gate and the various appurtenances connected with it.

All plans for the construction of the gate, hoisting machinery and other necessary features have been completed and a call for bids for its construction will be issued on about 1 February 1959.

Paper 1874 December, 1958 WW 5

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COLUMBIA BASIN STREAMFLOW ROUTING BY COMPUTER

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(Proc. Paper 1874)

ABSTRACT

A method for using a digital computer for streamflow routing in the Columbia Basin is described. Water excesses of rainfall or snowmelt can be routed by sub-basins to synthesize streamflow, which in turn is routed through lakes and channels to downstream control points. A new routing technique is made possible by use of the computer.

INTRODUCTION

Among the many types of engineering problems involving great masses of computations are those encountered in hydrologic practice for synthesizing streamflow. This paper describes a method developed specifically for deriving streamflow variation in the Columbia River Basin. The method was designed primarily for providing a direct and objective streamflow forecasting procedure for periods up to 10 days in advance, on the basis of preceding streamflows and forecasts of basin inputs from snowmelt or rainfall. It may also be used for design flood determinations or reservoir regulation studies. The procedure utilizes a new technique for streamflow routing which is made practical by the capabilities of a medium speed electronic digital computer. The primary capabilities of electronic digital computers, when applied to streamflow forecasting, are their ability to: (1) handle large amounts of input data, which are used to define the initial streamflow condition and forecasts of hydrometeorological events; (2) perform rapid arithmetic computations, for routing streamflow in small finite increments of time and storage; (3) follow a predetermined series of instructions, which automatically directs the arithmetic and data processing operation; (4) store numerous digital values

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representing basin, channel, or lake routing coefficients for use as directed by the basic program of instructions; and (5) provide digital output values successively through the operation, which are used for defining forecasts of streamflow at key gaging stations.

Program Development

The program was developed in the office of Division Engineer, U. S. Army Engineer Division, North Pacific, which supervises the planning and operation of several major water control projects in the Columbia River Basin. Daily reservoir regulation problems as well as project design computations require the use of a streamflow routing technique for the entire basin. The problem involves synthesizing streamflow for the component sub-basins lying within the Columbia River Basin, and routing streamflow through lakes, reservoirs, and channels in order to establish predictions of flow at a number of key downstream gaging stations. The essential requirements for this procedure are: (1) the time increment must be small enough to represent fluctuations of streamflow which occur in the Columbia River Basin tributaries (normally, 6 hours); (2) the total time for preparation of forecasts, from receipt of basic meteorologic and hydrologic data to derivation of forecasted flows should be less than four hours; (3) the method should provide for adjusting streamflow values periodically (usually daily) in accordance with observed conditions; and (4) the method of applying forecast values of input of water supply (either in the form of rainfall or snowmelt excesses, or streamflow) should be flexible, and allow for applying more than one forecast condition for any forecast period. During August 1956 the North Pacific Division Office obtained use of a type 650 IBM Electronic Digital Computer, with certain peripheral equipment, primarily for computation of system power studies for the Columbia River Basin. This computer is adaptable to the solution of streamflow routing problems in accordance with the requirements set forth above. The program enables the computer to solve the storage routing equations in many successive small increments, combine computed tributary and local inflows successively downstream, and provide values of routed flows at all specified gage locations. The operation is entirely automatic and is controlled by proper sequencing of input values. In effect, when all storage constants have been evaluated, the program provides a model of the Columbia River System which can be used to represent streamflow for any specified meteorological or storage condition.

The program has been checked out, and sufficient storage routing constants have been derived to test it on a day-to-day forecasting basis during the 1957 spring snowmelt flood. Streamflow forecasts were made on 35 consecutive days, which forecasts were used to help establish regulation of streamflow at reservoirs (primarily Grand Coulee Dam) for flood control operation on the main stem of the Columbia River. This trial showed the method to perform as desired, and results to be available within prescribed time limitations.

Columbia River Basin Storage

There are three types of storage delay to runoff which must be evaluated in the Columbia River System. These are: (1) basin storage, whereby the streamflow at an upstream gaging station can be computed as a time-rate

function from basin input values of rainfall or snowmelt excesses on the tributary areas; (2) lake storage, resulting from the many large lakes in major tributaries, whose effects on streamflow may be evaluated by the basic reservoir-type storage equation; and (3) channel storage, which results from the natural delay of flow through channel reaches. There are several commonly used methods for evaluating storage effects listed above. Except for lakes or reservoirs for which storage is a direct function of outflow, the methods are empirical in nature, in that they depend upon trial-and-error procedures for establishing or verifying basin or channel time-delay characteristics. Unit hydrographs and various channel routing techniques fall within this category.

Routing Method

The program developed as described herein relies upon a routing method which is basically in the form of the general storage equation

$$I_t = O_t + dS/dt \tag{1}$$

where I_t and O_t are inflow and outflow, respectively, in cubic feet per second, and $dS/_{dt}$ is the rate of change of storage at time, t. For cases where storage is a function of outflow (as in natural lakes, or for channel storage for short reaches where wedge storage is negligible in comparison with prismatic storage)

$$S = T_s O \tag{2}$$

where T_s is the proportionality factor between storage and outflow. Differentiating Eq. (2) with respect to time,

$$dS/dt = T_s(dO/dt) \tag{3}$$

Substituting this expression in Eq. (1), it becomes

$$I_t = O_t + T_s(dO/dt)$$

or

$$dO/dt = \frac{I_t - O_t}{T_s} \tag{4}$$

which represents the form of the storage equation used in this method. For natural lakes, the value of T_s is not constant, but it can be evaluated from the storage and outflow characteristics. This can be done by evaluating the differential of Eq. (2) with respect to h,

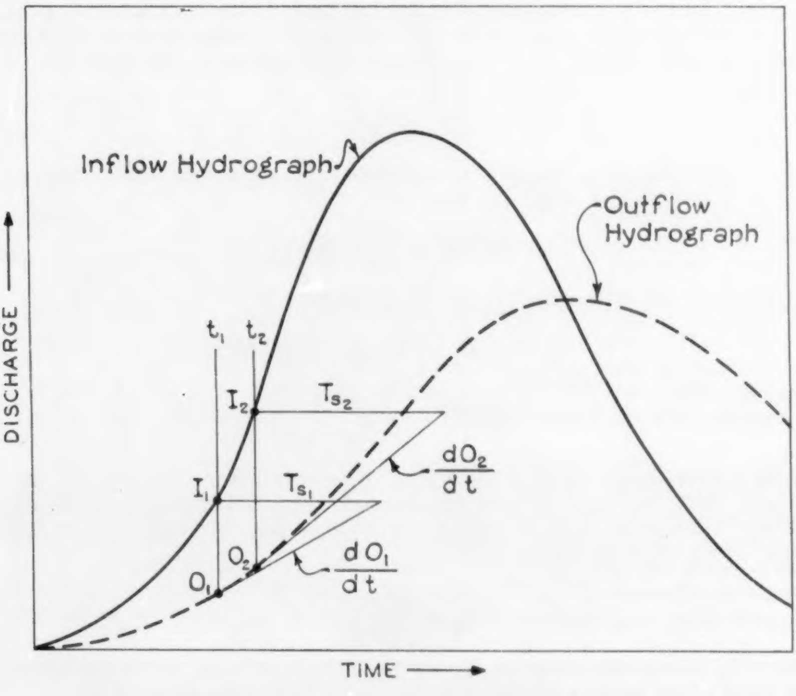
$$T_s = \frac{dS/dh}{dO/dh}$$

where T_s for a given elevation, h, is given in units of time, dS/dh represents the slope of the storage-elevation curve, and dO/dh is the slope of the discharge-elevation curve at elevation h. From Eq. (4) it can be shown that with zero inflow, the outflow recession is in the form

$$O_t = O_0 \epsilon^{-t/T_s} \tag{6}$$

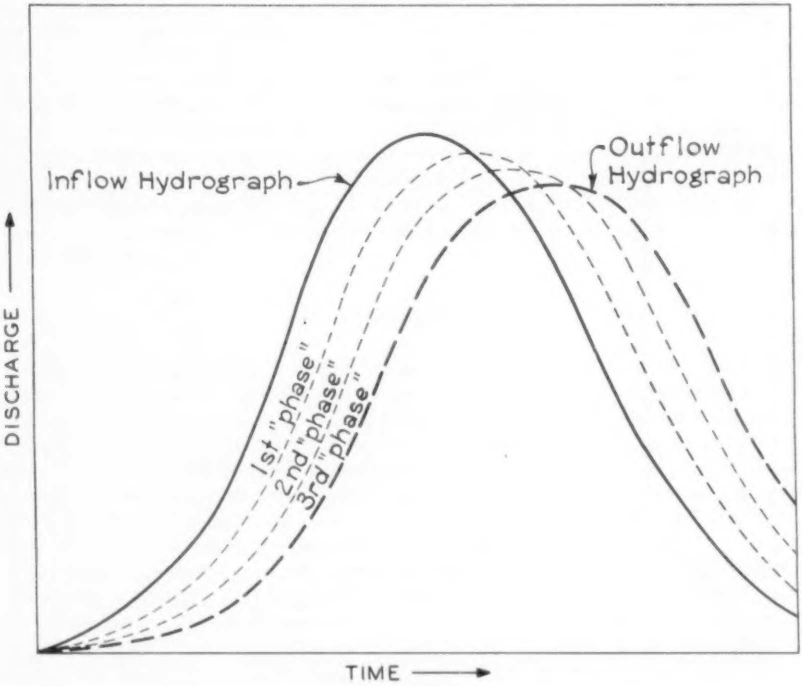
where O_0 is the initial outflow at time, t = O; O_t is the outflow at time, t; and T_s is the proportionality constant defined above, corresponding to the value of O_t. Eq. (6) is the typical decay-type function characteristic of streamflow recession, but with a varying recession coefficient for this method.

In application to natural lakes or reservoirs, Eq. (4) may be used in the program in small finite increments of time, as shown schematically in Fig. 1. The same general equation may be applied to channel storage by reducing the length of the channel reach to the point where "wedge" storage is negligibly small in comparison with "prismatic" storage. By successive routings (herein called "phases") through many small increments of reservoir-type storage, the time delay from channel storage can be evaluated as shown in Fig. 2. Similarly, basin storage can be evaluated through successive "phases" of reservoir-type storage, to represent empirically the effect of basin storage. This distributes runoff from basin input values in inches per 6-hour period, which provides streamflow in terms of cubic feet per second, comparable to that obtained through use of unit hydrographs. The storage routing method used herein has the advantage of being able to adjust values to observed



LAKE STORAGE EVALUATION

FIG. 1



CHANNEL STORAGE EVALUATION

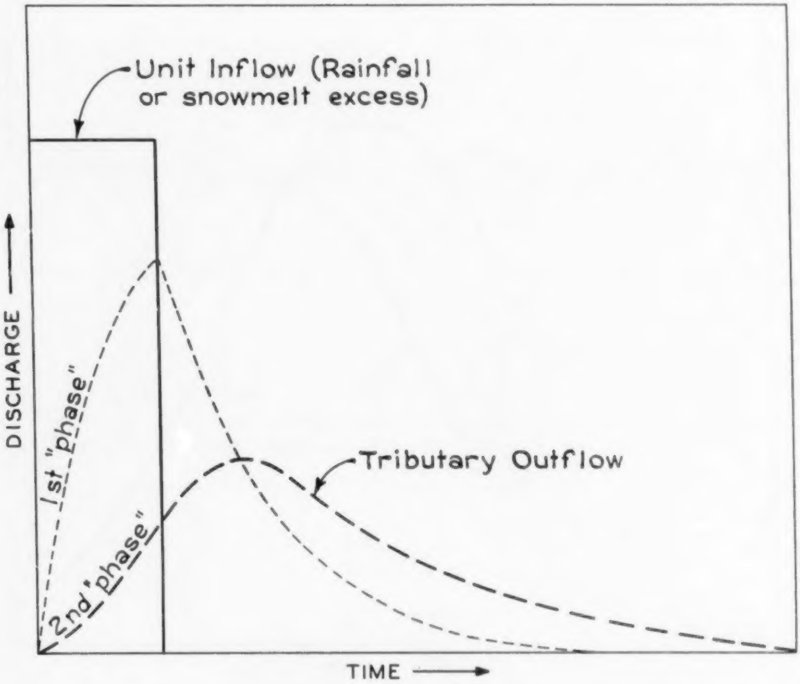
FIG. 2

conditions on a day-to-day basis. Fig. 3 shows schematically the distribution of runoff by successive routings of a unit basin input through 2 "phases" of reservoir-type storage.

An important feature of the routing method for basin and channel storage is that the time of storage may be varied with flow. This, in effect, results in distribution of runoff from snowmelt or rainfall excesses which may be varied with the flow condition. For example, the time delay to runoff may be made shorter with high rates of runoff and longer with low rates of runoff. Similarly, the time of travel of flood waves through channels may be made to vary with discharge, according to channel conditions. This feature allows great flexibility in empirical fit of data from constants developed from actual record, and the method may be used to represent closely the actual time delay to runoff.

Basic Codes and Routines

The basic code comprises a set of instructions which directs the computation of streamflow from input data read from punch cards, through



BASIN STORAGE EVALUATION

FIG. 3

appropriate routing sub-routines and summing routines. The three routing sub-routines, for evaluating basin, channel, and lake storage, are based on evaluation of the storage equations as explained above. The sub-period time increment for each phase of routing is three hours. Constants which are proportional to each of the storage times for each successive routing are used and stored for each basin area or channel reach. For evaluation of basin storage, inflow amounts are divided between surface (direct) and subsurface (groundwater) runoff, and each component is routed separately through two phases of reservoir storage.

All necessary constants for basin, channel, and lake storage for the entire basin, together with the necessary programs and routines for the actual routing, are contained on a single loading of the storage drum of the IBM 650, (2000 ten-digit word storage capacity). Sufficient space is reserved for obtaining outflow data for a 10-day forecast, with values of discharge given for four times per day (at 6-hour intervals). There are 68 stream gaging locations or local inflow points, for which discharge data can be obtained. Input cards may be read in for basins, with values given as snowmelt or rainfall excesses in inches per 6-hour period, and output cards are punched out which give values of streamflow in cubic feet per second for the stream gaging point.

This operation is repeated successively downstream, and tributary inflows are added and routed through channel or lake storage to the downstream points. A set of "pilot" routines directs the operation from one reach to the next, and ties the entire routing into a completely automatic operation. Thus, successive inflows are routed and summed from the headwaters of the basin to the outflow for Columbia River at The Dalles, Oregon. For a five-day forecast, flows for the entire basin can be routed in 3/4 of an hour of IBM 650 machine time. Fig. 4 is a Schematic Master Flow Diagram of the Columbia River Basin, showing the relative locations of the sub-basin areas, major lake storages, and channel reaches which are presently incorporated in the program.

Input Data

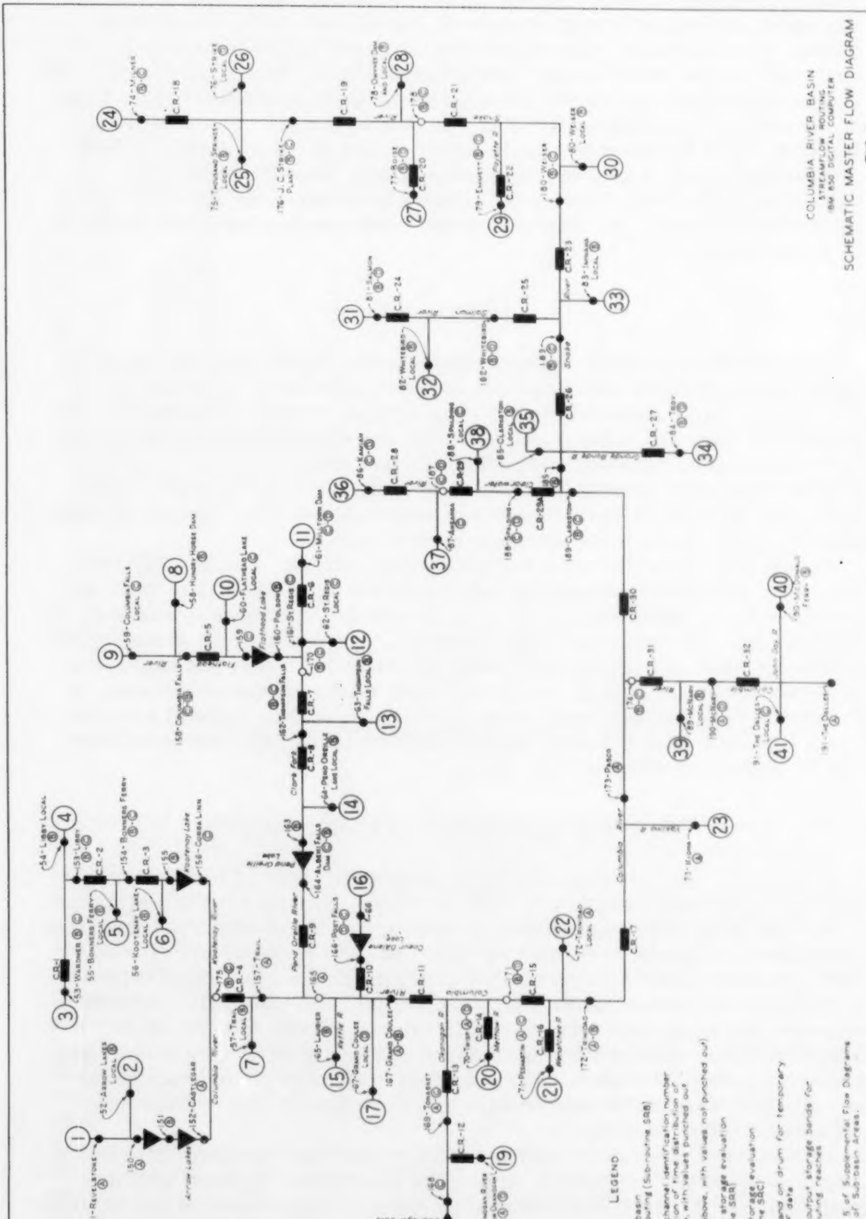
For any drainage basin or local ungaged area, inflows may be read in as basin runoff excesses resulting from snowmelt or rainfall, in inches per 6-hour period, or as basin outflows in cubic feet per second. In addition it is possible to enter any tributary at one of the downstream gaging points, with given or assumed values of streamflow in cubic feet per second, without necessitating routing through the tributaries upstream from the control point. Thus, it is possible to route flow in any section of the river from given natural or regulated flows at the upstream control points.

Snowmelt is computed from basin snowmelt indexes. For the 1957 trial operation, a simple maximum temperature index of snowmelt was used, with provision for an effectiveness factor to account for variations in albedo of the snow surface. Reference is made to chapters 5, 6, and 9 of the Summary Report of the Snow Investigations,(2) which describe the theory and application of snowmelt indexes. Estimated or observed values of snow cover area, in per cent of total drainage area at the gage, are required. Rainfall amounts, which may contribute to runoff during snowmelt floods, are estimated from reporting network observations.

Derivation of Basin Storage Coefficients

The routing method outlined above requires derivation of basin storage routing coefficients, similar in function to unit hydrographs. For application to the Columbia River Basin, the snowmelt inputs are divided between surface and subsurface runoff, in the ratio of 20% and 80%, respectively. The computer program allows selection of any arbitrary division of inflows between surface and subsurface runoff that is desired for each sub-basin. In order to represent the basin time delay to runoff, two coefficients (one for each "phase" of reservoir-type storage routing) are required for both surface and subsurface runoff evaluation. The two coefficients are able to describe the rate-of-flow variation for each component, just as many unit hydrography ordinates will do. (See Fig. 3).

The ability of program to vary the storage time with discharge provides a simple and realistic method for varying the distribution of runoff with flow conditions. C. O. Clark(3) recognized the need for such variation and suggested methods for altering the shape of unit graphs, and thereby provide for a sharper and earlier peaked unit hydrograph for high rates of runoff than would be used for low rates. The variation of storage time with discharge for the



COLUMBIA RIVER BASIN
STREAMFLOW ROUTING
IBM 650 DIGITAL COMPUTER
SCHEMATIC MASTER FLOW DIAGRAM
FIG 4

method described herein results in a distribution which can be varied according to any arbitrarily assumed relation between storage time (T_s) and discharge. For synthesis of streamflow in the Columbia Basin, it was assumed that the T_s was related inversely to the 0.2 power of the discharge. This can be expressed mathematically by the relationship

$$T_s = KQ_t^{-.2} \tag{7}$$

wherein T_s is the storage time (in 6-hour periods) for a particular phase of reservoir-type storage routing, K is the basin storage coefficient for that phase, and Q_t is the outflow at time, t, in cubic feet per second. Values of K for each "phase" of routing are stored in the memory drum of the IBM 650, for use as required in the routing for each sub-basin. K_1 and K_2 are used to represent surface runoff coefficients, and K_3 and K_4 represent subsurface runoff coefficients.

Fig. 5 illustrates the varying unit distribution of runoff at different discharges for a hypothetical 3790 square mile drainage basin, with values of K_1 and K_2 equal to 20.0, and K_3 and K_4 equal to 40.0. Curve "A" in Fig. 6 represents the outflow resulting from an inflow of 0.5 inch in the first 6-hour period, with an initial outflow of zero. Curve "B" shows the outflow resulting from the case where the initial flow was stabilized at the rate of 0.5 inch per day (equal to 50,000 cfs). The inflow during the first 6-hour period was increased by 0.5 inch, after which the inflow was reduced to the 0.5 inch per day rate for the remainder of the time. The difference in distribution between the two cases shows the relative change in storage characteristics for low and high flow conditions that can be incorporated in the method. Between such extremes, the variation in storage time is a continuous function, as set forth in Eq. (7).

Fig. 6 shows the computed outflow hydrographs that would result from 10 days of continuous snowmelt, with an initial flow of zero. The inflow was made to vary in a daily fixed percentage pattern of snowmelt, in order to account for its diurnal variation. The resulting outflow hydrographs were computed using three different sets of basin routing coefficients, as shown in Fig. 6. These curves illustrate how the basin storage coefficients may be varied to represent the runoff characteristics of a given basin. They also show that the diurnal fluctuation in streamflow resulting from daily snowmelt variation can be reproduced by this method. With greater storage delay, this effect tends to be "dampened" out. The outflow hydrographs in Fig. 6 are similar to S-curves developed from unit hydrograph computations. The diurnal fluctuations should not be confused with the fluctuations that sometimes occur from S-curve computations, resulting from incorrect proportionment of unit hydrographs.

Actual values of basin routing coefficients are determined by trial and methods in reconstituting observed conditions of streamflow. Reconstitutions thus far have been limited to sub-basins of the Columbia River, for which snowmelt is the principal source of water supply. The computer program also makes provision for adding base flow amounts to the derived runoff from surface and subsurface flow. The routing technique has been tested sufficiently to show its feasibility and acceptable accuracy of results. For streamflow forecasting, the method has the distinct advantage of being capable of easy adjustment to observed flow conditions on a day-to-day basis. This would be extremely cumbersome using unit hydrograph procedures.

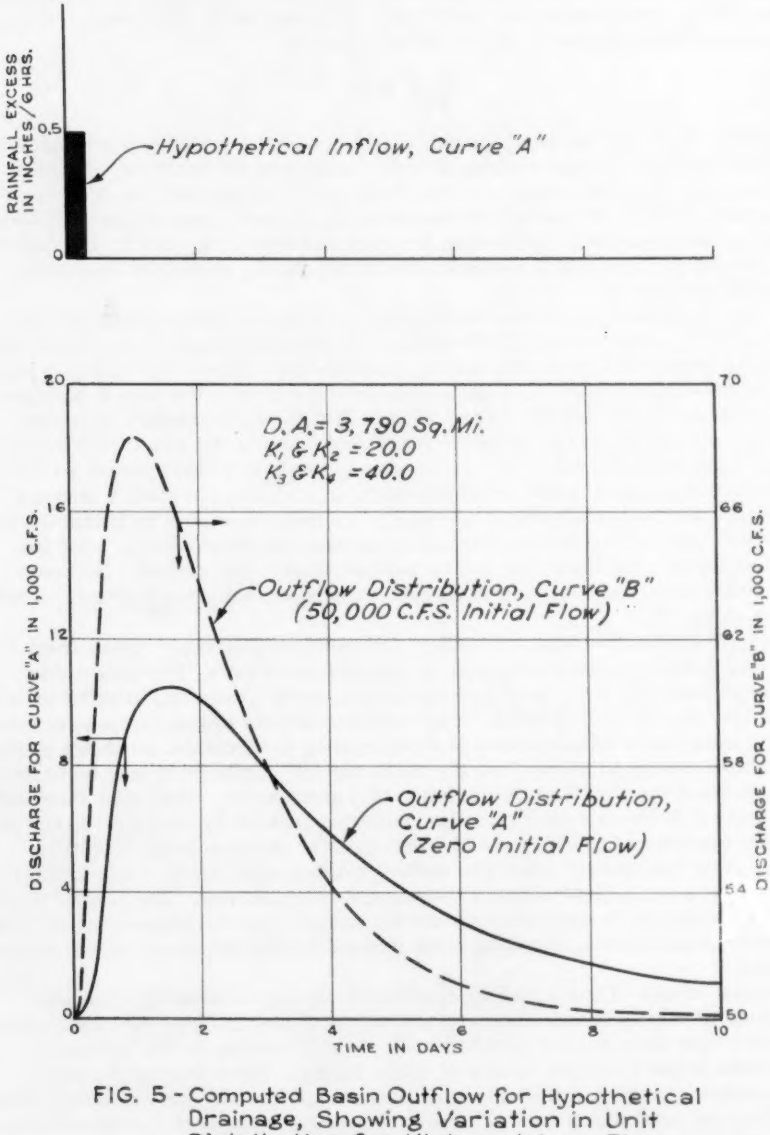


FIG. 5 - Computed Basin Outflow for Hypothetical Drainage, Showing Variation in Unit Distribution for High and Low Flow Conditions

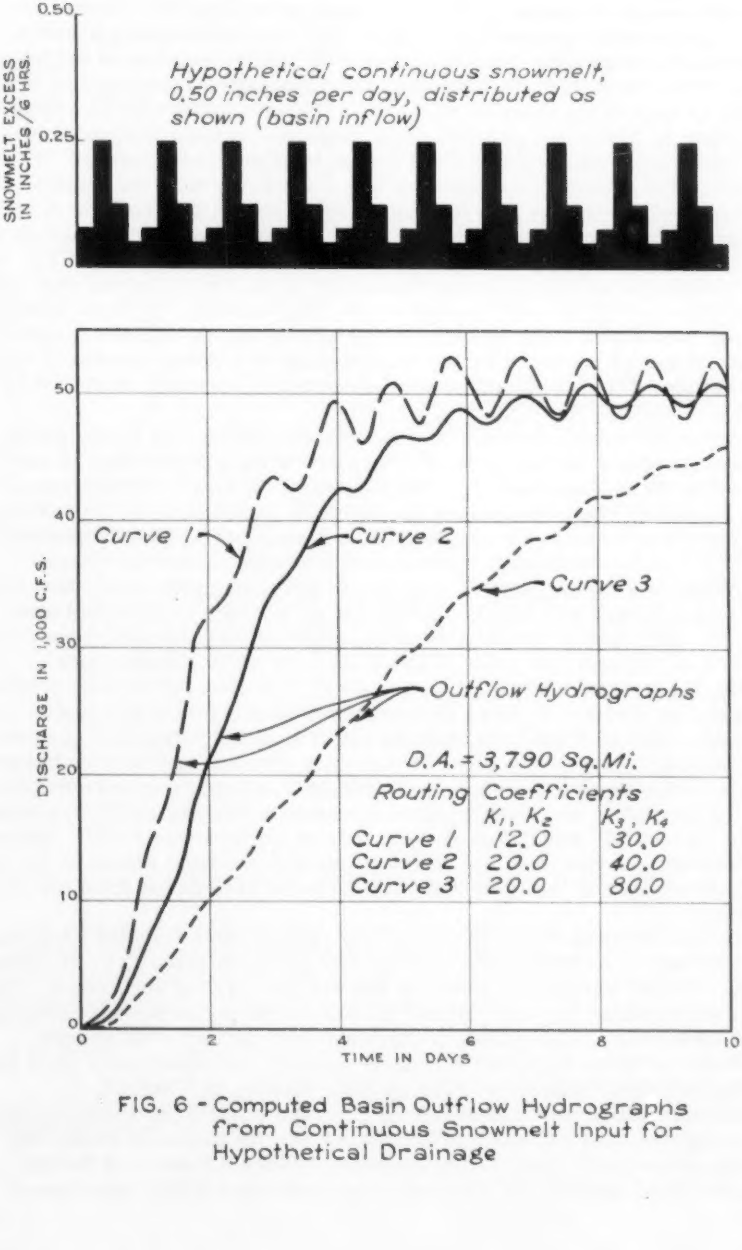


FIG. 6 - Computed Basin Outflow Hydrographs from Continuous Snowmelt Input for Hypothetical Drainage

Derivation of Channel Storage Coefficients

Having developed the concept of using many small segments of reservoir-type storage to represent the storage effect of natural stream channels, it is then necessary to develop methods for applying it to specific channel reaches. It is convenient to consider each channel reach as representing a certain amount of storage time (usually in hours) for which a flood wave is transposed by traveling through it. In addition to the transposition, there is also modification of form of the flood wave, by which the wave (or any time irregularity of inflow) is "flattened" or modified in shape with respect to time. In the commonly used "Muskingum" flood routing equation,(4) the value of "K" determines the amount of transposition of a flood wave, while the value of "x" (the ratio of the storage as a function of inflow to that as a function of outflow) determines the modification of wave form. The Muskingum "K" value is expressed in hours, and where "x" = 0, it is equivalent to the value of T_S. The modification of wave form in the Muskingum method is dependent upon a derived value of "x" for each channel reach. The limits of "x" range from 0.5 for the case where no modification occurs, to 0.0 for the maximum modification of wave form which occurs when storage is a direct function of outflow. The values of "x" for natural stream channels are normally between 0.20 and 0.35.

In multiple-phase reservoir storage channel routing, the same general technique applies, in that the total transposition for a given reach is represented by the storage time, T_S. The number of "phases" of reservoir storage in a particular reach determines the degree of modification of the wave form. With only one "phase", the modification is a maximum and corresponds to use of "x" = 0 in the Muskingum equation. With a large number of "phases", equivalent to a routing time of, say, 1 hour per phase, pure translation of the flood wave results and is equivalent to use of "x" = 0.5 in the Muskingum method. Intermediate between these extremes, any number of "phases" may be used to represent the wave modification for a given channel reach.

Fig. 7 represents the variation in wave modification that may be obtained through use of this technique. An assumed arbitrary inflow was routed through a total of 36 hours of time, by use of 1, 3, and 6 "phases" of reservoir storage, respectively. It is seen that the greatest modification occurs with a single "phase" of 36 hours. (Curve "A"), which shows the effect of routing through an uncontrolled lake or reservoir with that particular storage time. Curves "B" and "C" show the computed outflows with 3 and 6 "phases", respectively. In this case, the use of 3 "phases" produced results which were equivalent to use of "x" equal to about 0.35 in the Muskingum flood routing equation.

For the Columbia River Basin, channel reaches were selected from gaging stations and key control points, and they are generally from 20 to 50 miles in length. Three "phases" of reservoir storage are used for each reach. The mean storage time for each "phase" of channel routing on the main stem is about three hours. The routing coefficients were derived from general knowledge of travel times in the channel reaches, and adjustments were made by trial and error procedures through reconstituting past record.

The sub-routine for channel routing also incorporates the ability to change the storage time as a function of discharge. For the Columbia River, the storage time was allowed to vary inversely with the 0.2 power of the discharge. Thus, doubling the discharge results in about a 13% reduction of

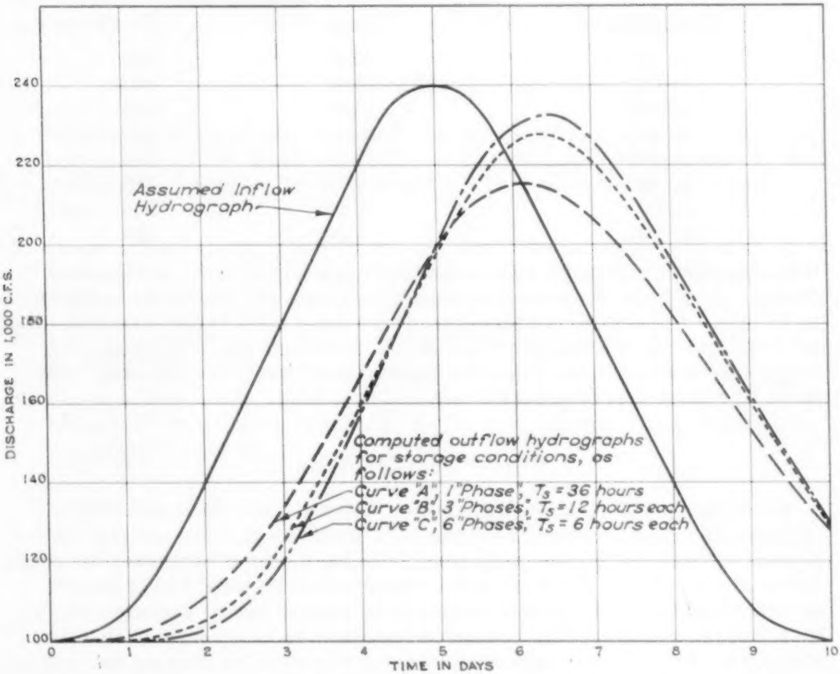


FIG. 7 - Channel Storage Evaluation for Hypothetical Inflow, Showing Evaluation of Outflow Through Varying Numbers of Routing Phases

storage time. The program is flexible and would allow any arbitrary function of storage time with discharge. The computer operation of "Table Look Up" is used for this purpose. This technique also has the advantage of doing away with the unrealistic "dips" in outflow resulting from a sudden rise in inflow, which is characteristic of the Muskingum method.

Derivation of Lake Routing Coefficients

Lake routing coefficients were determined directly, by evaluating T_s through use of the relationship given in Eq. (5) for a number of elevations. This establishes the relationship between storage time and discharge. For the computer program 25 such values were determined for each lake to cover the range of flows normally experienced. "Table Look Up" operation was used to select the value of storage time corresponding to a given outflow in the routing sub-routine. The following tabulation illustrates the variation of storage time (T_s) with discharge for Kootenay Lake, which is one of the major lakes in the Columbia Basin, in units of either days or hours:

Discharge, cfs	Storage Time Days	Storage Time Hours
10,000	20.6	495
20,000	16.8	402
30,000	13.9	334
40,000	11.2	268
50,000	10.2	244
70,000	9.16	220
100,000	8.53	205
150,000	6.24	189

It is seen that the storage time decreases markedly with increasing discharge. It will also be noted that the storage time of this lake is in the order of 10 days. The channel storage time in the main stem of the Columbia River is in the order of 6 miles per hour, or 140 miles per day. Obviously, the major lake storage in the Columbia Basin constitutes a much greater time delay to run off than channel storage.

Supplementary Routines

In addition to the main routing routine with its subroutines and pilot routines, two supplementary routines were written which are entirely independent. These are for the purpose of reducing the time necessary for preparation of a forecast. The first is the "distribution routine" and is used to distribute daily values of basin snowmelt or rainfall runoff excesses into 6-hour increments, according to a percentage pattern which may be changed with time. The second supplementary routine is used for plotting the outflows in cubic feet per second for the end of each 6-hour period, in the form of hydrographs, on the IBM 402 tabulating machine.

SUMMARY

The purpose of this description is to present the overall scope and basic routing method for the procedure. The many details of programming and coding, as well as the data processing techniques required to provide a workable solution in the time normally available for streamflow forecasting, have not been mentioned. The method provides the framework for completely automatic determination of the components of streamflow in the Columbia River Basin, whereby all hydrometeorological factors affecting runoff may be evaluated objectively through cause and effect relationships. The method is feasible only with an electronic computer, but with the general advent of computers, it provides the hydrologist with a flexible and rapid technique for synthesizing streamflow.

ACKNOWLEDGMENT

This procedure was developed under the general supervision of Mr. Mark L. Nelson, Head, Water Control Branch, U. S. Army Engineer Division, North Pacific. The author wishes to acknowledge the work done by Mr. Edward M. Davis of that office in coding the basic program and supplementary routines,

and for general aid in data processing techniques. His work contributed immeasurably to the successful completion of the procedure.

REFERENCES

1. Rockwood, David M. and Carver E. Hildebrand, "An electronic analog for multiple-stage reservoir-type storage routing", Corps of Engineers, Civil Works Investigations, Project CW-171, Technical Bulletin 18, March, 1956.

2. Corps of Engineers, North Pacific Division, "Summary Report of the Snow Investigations, Snow Hydrology", 30 June 1956, U. S. Army Engineer Division, North Pacific, Portland, Oregon.

3. Clark, C. O., "Storage and the unit hydrograph," Trans. ASCE, Vol. 110, pp. 1419-1446, 1945.

4. Corps of Engineers, Chief of Engineers, "Routing of floods through river channels," Part CXIV, Chapter 8, Engineering Manual, Civil Works Construction, September 1953.

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Journal of the
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Proceedings of the American Society of Civil Engineers

FINANCING OF SAND BY-PASSING OPERATIONS

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(Proc. Paper 1875)

SYNOPSIS

This paper presents information which was available in mid-1958 on the financing of twelve sand by-passing operations. Comments are made as to possible trends in the financing of work of this kind, and information is given on the problems and procedures involved in financing sand by-passing projects at various governmental levels.

INTRODUCTION

In addition to determining where sand by-passing should be done, and how it should be accomplished, the engineer is often called on to plan or assist in planning the financing of the project.

Sand by-passing projects are often large and expensive, and their financing may be complex. Sand by-passing is also a relatively new type of operation; thus experience has not established firm guide lines, especially applicable to this type of work, for the help of the financial planner. A review of the financing of the limited number of past, present and proposed projects on which such information is available, will nevertheless be of assistance to the engineer newly confronted with the problem. Such a review will point out details which seem to indicate trends thought to be in progress with respect to the financing of operations of this kind, will provide information on how the usual methods of financing public works have been applied to sand by-passing, and will point up the steps which, if observed, are likely to be of importance in working out the complete financial program.

A summary with regard to the financing of twelve sand by-passing operations is shown in Table 1. These operations include all those on which financial information was readily available in mid-1958. The year indicated in

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1. County Engr., West Palm Beach, Fla.

TABLE 1. - FINANCING OF SAND BY-PASSING OPERATIONS

(Amounts are in thousands)

LOCATION	DATE	COST		SOURCE OF FUNDS				
		CAPITAL	PERIODIC	USA	STATE	CO	CITY	OTHER
Santa Monica, Cal.	1949		250		125[a]	--	---	
Port Hueneme, Cal.	1953		1,838	1,838				
Rudee Inlet, Va.	1955	60[b]		---	---		---	
Santa Barbara, Cal.	1956		90	30		26	34	
Santa Monica, Cal.	1957		300		300			
Lake Worth Inlet, Fla.	1958	550		100		20[c]	430	
Shark River Inlet, N.J.	1958		220		220			
San Cruz, Cal.		337		219				118[d]
Camp Pendleton, Cal.			300	300				
Ventura Co., Cal.		2,900		2,900				
Ventura Co., Cal.			800	800				
Jupiter Inlet, Fla.	1957[e]		30			10[f]		20[g]

a Plus $125,000 of combined city and county funds.

b Capital costs, and annual operation and maintenance cost of $20,000, are financed out of combined Federal, state and city funds, some of which were raised by general obligation city bonds.

c This cost, plus annual operation and maintenance cost of $20,000, paid by county from special district tax.

d Paid by Santa Cruz Port District, which will also cooperate in paying annual operation and maintenance cost of $31,000.

e This periodic operation was also performed several times previously.

f Paid for by county from special district tax.

g Paid by Jupiter Inlet District.

the table is that in which the operation, if periodic, began; otherwise it is the year in which continuous operation began. Where no year is shown, the project is a proposed project. Capital and periodic costs are listed, together with the sources of funds. The following comments are offered with respect to the individual projects:

Santa Monica, California

This operation consists of removing an accretion of sand from the beach behind the breakwater at Santa Monica, and filling other beach frontage with sand where erosion has taken place. It is done periodically and has been carried out twice, once in 1949 and again in 1957. It will be repeated as necessary in the future. The work is performed with floating equipment, and some 500,000 to 600,000 cubic yards of material are moved in each operation. The first operation in 1949 cost about $250,000. The State of California provided half of this amount, and the remainder was provided jointly by Los Angeles County, the City of Los Angeles and the City of Santa Monica. The next operation, in 1957, cost about $300,000 and was financed entirely by the State.

Note is made of the fact that several agencies joined in financing the first operation, whereas eight years later at the time of the second operation the State assumed the responsibility for paying for the entire job.

Port Hueneme, California

The operation at Port Hueneme involved the moving of about 4,000,000 cubic yards of material to correct serious erosion downdrift from Port Hueneme and at the United States Navy installation at Point Mugu. It was carried out in 1953 and 1954 with floating equipment, at a cost of approximately $1,838,000. The funds were provided by Congress.

The entrance to Port Hueneme has the effect of diverting the littoral drift out into deep water from which it is not recovered. Thus it is lost to the downdrift shores in which the Government has an interest. The Government, therefore, financed the entire sand by-passing operation to protect and restore Government property as well as other property its work had damaged.

Rudee Inlet, Virginia

The construction and operation of the Rudee Inlet sand by-passing plant is one of the features of a larger program for the restoration and maintenance of beaches and ocean front improvements at Virginia Beach. Finances for the entire program total about $1,600,000, consisting of $233,000 provided by the Federal Government, $240,000 provided by the Commonwealth of Virginia at the rate of $40,000 per year for six years, and $1,127,000 provided by the City of Virginia Beach, $900,000 of which was raised through a general obligation bond issue. Out of the total fund, the City periodically appropriates and transfers funds to the Virginia Beach Erosion Commission for use in the work. The sand by-passing plant at Rudee Inlet was financed by such a transfer, and cost about $60,000 to build in 1955. It is a fixed dredge with an intake at the end of a movable boom, and pumps about 42,000 cubic yards of sand per year onto the downdrift beach. The annual operating and maintenance cost is approximately $20,000, and is furnished from the City's General Fund.

Like the first operation at Santa Monica in 1949, funds were obtained from several sources for the Virginia Beach project. Federal funds were obtained on the basis of the Government's approval of the project as one which would correct the erosion of publicly owned beaches, Commonwealth of Virginia funds were provided under the laws of that state, and city funds from a bond issue as well as from direct taxation went into the work.

Santa Barbara, California

This work is a continuing program in which some 300,000 cubic yards of sand are pumped annually from the Santa Barbara harbor entrance to the downdrift beach. Floating equipment is used, and the City reports the annual cost at $90,000. Of this amount, the Federal Government furnishes $30,000 from harbor maintenance funds, Santa Barbara County provides $26,000, and $34,000 is furnished from the City of Santa Barbara harbor budget. Harbor budget funds come from general property taxes as well as franchises, rentals, and the like.

Here again is demonstrated the joint financing of a sand by-passing project from several sources. Federal funds in this case arise from the project's inclusion of harbor maintenance dredging as one of its features, and city and county funds come from direct taxation and from various harbor revenues.

Lake Worth Inlet, Florida

The installation is on the north jetty at Lake Worth Inlet, and consists of a fixed dredge with an intake at the end of a movable boom. It is expected to pump about 100,000 cubic yards of sand per year onto the downdrift beach to the south, through a submerged pipeline anchored to the bottom of the Inlet. The capital cost of the installation was about $550,000, of which some $530,000 was furnished by the Town of Palm Beach, and $20,000 by Palm Beach County. At a later date the Federal Government is expected to furnish a reimbursement of about $100,000, which will have the effect of reducing the Town's outlay to $430,000. This $430,000 consists of $400,000 from a general obligation bond issue, plus a little over $25,000 from general property taxation, plus a few thousand dollars of interest that was earned by investing the construction funds in short term Government securities pending their use. The County's share, $20,000, was obtained from general property taxation in a special district fronting the ocean throughout the County. Operating and maintenance costs are expected to total about $20,000 per year, and will also be furnished by the County from taxes levied in the same special district.

Lake Worth Inlet is another case of the cooperative financing of this type of work. Federal participation was made possible because of savings expected by the Government in harbor maintenance costs, and because the installation is part of a larger program of work approved by the Government for restoring the County's beaches, some of which are publicly owned. Local funds came principally from general obligation bonds, with a lesser amount from direct taxation; in both instances supported by the areas along the ocean front most directly benefitted by the work.

It is further noted that Lake Worth Inlet is a case where the local authority, in agreeing originally with the Federal Government for the improvement of an inlet, relieved the Government of responsibility for correcting damages to the coast that might result from the work. Thus, while it is now generally conceded that the deepening of Lake Worth Inlet has been a major factor in cutting off the littoral drift and starving the beaches to the south, the Government is not obligated to correct the situation. Government participation is instead based only on harbor maintenance savings, and the normal percentages and allowances authorized by law for beach restoration in general.

Shark River Inlet, New Jersey

During the period October 1, 1958, to October 1, 1959, a dragline and trucks will move and haul about 250,000 cubic yards of sand from Shark River Inlet, Monmouth County, New Jersey, and deposit the material so as to feed the depleted beach to the north. The work will cost about $220,000 and will be paid for entirely by the State of New Jersey.

Santa Cruz, California

A plan for a jettied small boat harbor in Wood's Lagoon on the east side of Santa Cruz includes provisions for sand by-passing in the event such an operation proves necessary. A floating dredge will be used, with a combination of floating and fixed discharge lines to place the material in proper position to nourish the downdrift beach. Data as to the quantity of material it may be

necessary to by-pass are inconclusive, and estimates range from 25,000 to 300,000 cubic yards per year. The capital cost of the equipment will be about \$337,000, of which \$219,000 will be furnished by the Federal Government, and \$118,000 by the Santa Cruz Port District. It is also understood that the Port District will cooperate in the operation and maintenance cost, which is estimated at \$31,000 per year. In this case, as distinguished from that at Lake Worth Inlet, it appears that the Federal Government may be assuming a considerable degree of responsibility for correcting beach erosion brought about by the construction of an inlet.

Camp Pendleton, California

A periodic operation has been planned in which about 500,000 cubic yards of sand will be pumped from Camp Pendleton Harbor and its entrance every five years, and deposited so as to replenish the beaches fronting the City of Oceanside. The work is to be done with floating equipment, and is estimated to cost about \$300,000 per operation. It will be paid for by the Federal Government through Congressional appropriations at the rate of \$60,000 per year for the maintenance of Camp Pendleton Harbor. This is an example of the Government combining maintenance operations with the restoration of the beaches that were damaged by Government work.

Ventura County Small Craft Harbor, California

A plan has been developed to construct a small craft harbor, involving navigation channels and a sand entrapment area, near Port Hueneme. The material from the construction will be pumped to the beach downdrift from Port Hueneme, and thereafter at two year intervals the sand from the entrapment area will also be pumped to the same beach. Floating equipment will be used for the work. The original construction will involve some 5,200,000 cubic yards of material at a cost of about \$2,900,000, and the periodic operations every two years will each involve about 1,600,000 cubic yards at a cost of \$800,000. The Federal Government will pay the initial cost of \$2,900,000, and will also pay the periodic cost of pumping out the entrapment area, the latter from Congressional appropriations of \$453,000 made annually for the maintenance of the project. This project will provide additional and continuing nourishment of the beaches downdrift from Port Hueneme, which were first replenished in 1953-54.

Miscellaneous Projects

There are probably many sand by-passing projects of small to moderate size that are not thought of as sand by-passing projects and consequently remain unnoticed. One of these, which is mentioned only as a typical example, is at Jupiter Inlet, Florida. This is a shallow inlet which needs dredging every two years. The affairs of the Inlet are managed by an elected commission which assesses property taxes in a special district to pay for inlet maintenance. The biennial job involves about 70,000 cubic yards of sand and costs about \$30,000. Palm Beach County contributes \$10,000 of this amount, for

which it is able to have the sand pumped onto the badly depleted beach to the south, instead of to the closest convenient spoil area.

CONCLUSIONS

The preceding examples indicate that funds for sand by-passing may be available from federal, state, county or city sources, or from various types of special districts. The examples also offer evidence of a trend, thought to be in progress, toward increased recognition of beach erosion as a state and national problem. About half of the listed projects are financed solely with state or national funds, and state or national funds are included in the financing of most of the rest. It is usual, however, for several local agencies such as counties, cities or special districts, to cooperate jointly in projects that involve local funds in considerable amount. The following additional comments are offered with respect to the various sources of financing:

Federal funds for construction and maintenance are made available to public agencies, within the discretion of Congress, on the basis of studies and reports showing that the benefits to be derived from the work will exceed the costs. The Federal Government participates up to fifty per cent in such studies and reports, which are usually made by the United States Army District Engineer in whose district the project is located. In a favorable report, recommending that the work be done, Federal Aid may be recommended according to law in a proportion equal to one third of the percentage of the beach that is publicly owned, plus limited allowances for indirect public benefits to beaches not publicly owned, plus allowances for any money the Government will be able to save (such as for reduced harbor maintenance) because of the construction of the project. Where sand by-passing operations are needed to maintain, protect, or restore Government property, the Government may pay the entire cost, notwithstanding non-Government property may also benefit from the work. Where sand by-passing is needed to restore and maintain beaches not owned by the Government, but which are depleted because of Government work, the Government may likewise, in its discretion, pay the entire cost. However, in this case, if the Government has been relieved by the local authorities of responsibility for correcting the damage caused by its work, it may pay only the percentages and amounts prescribed by law as previously outlined. Early coordination should be effected with the United States Army District Engineer with reference to Government financing, and should in fact be effected in any event, to take advantage of Government assistance other than financial, and to enable compliance with applicable Government regulations.

The financial assistance rendered by the various states ranges from none at all to paying one hundred per cent of the cost of the project. The United States Army District Engineer will be able to give information as to what assistance the particular state offers, and what agency or official has charge of the program. Coordination with the state may thus be established, confirming the scope of available assistance, determining what qualifications must be met to be eligible to receive it, and proceeding to the inclusion of that assistance in the sand by-passing project under consideration.

Counties are also often concerned with sand by-passing projects and involved in their financing, though the authority and responsibility of the county to engage in such work varies from state to state, and may vary to some extent within a given state. County governments, however, are for the most part

relatively flexible, and also highly sensitive to public need. It is therefore possible for them to adjust themselves with reasonable promptness to situations that confront them. For this reason the County Attorney, or other officer or employee who represents the county in civil law matters, will not only be of great help in outlining the county's existing authority to participate in the cost of a sand by-passing operation, but will also be able to furnish guidance as to new legislation or other proper means for having that authority modified, in the event its modification is desired by the local authorities to better fit the situation at hand.

Funds available directly from taxation at the county level might usually be sufficient for modest participation in capital or periodic sand by-passing costs, and will generally be sufficient for the operation and maintenance of a fixed dredging installation. On the other hand, major county participation in the cost of one of the larger and more expensive operations is likely to involve bond financing.

It will be advisable, in the early stages of any bond program, to engage the services of skilled fiscal agents, who will develop and document the extensive information needed to meet legal requirements and show the financial soundness of the bonds being offered, and who will market the issue. General obligation bonds will be the type usually issued, and prior to such an issue it is probable the law will require that a referendum be held to secure the approval of the taxpayers who will be called on to pay off the debt. In addition, the issue will have to be approved by bonding attorneys recognized in the financial world for the reliability of their opinions on the validity of bonds, and will have to be validated by such courts as the bonding attorneys designate.

For a project such as the restoration of a popular resort beach of the type where income from concessions and recreational facilities is considerable, it may be realistic to consider revenue bonds for financing purposes instead of general obligation bonds. A revenue issue presents the same problems as a general obligation issue, except that a referendum may not be needed, and the fiscal agents will not be primarily responsible for developing the information needed to show that the bonds are financially sound. Instead, a feasibility report, prepared by a firm recognized in the financial world for the reliability of its reports on the financial soundness of revenue projects, will be required to assure a satisfactory market for the issue.

A badly eroded beach is generally a more serious difficulty to a city than to other levels of government, and city funds are thus repeatedly found in the financial frameworks of sand by-passing projects. City charters usually provide considerable latitude as to governmental functions and operations, and the charters of coastal cities commonly and fortunately provide this latitude with respect to work on the ocean front. Thus the city can almost always find a way to assist to some extent in sand by-passing work for which it recognizes a need. The City Attorney will be able to furnish information as to the procedures involved in making city funds available; and, like the County Attorney, the City Attorney will also be able to guide the way toward any new legislation that may be needed to enable the city to cope with its problem most effectively.

The city, like the county, is generally limited as to the amount it can raise at one time by direct taxation, and it may likewise have to sell bonds to finance sand by-passing operations of major size. The city's problems, and the steps it must go through in any bond program, are similar in all respects to those of the county, and the same services will be required.

Special districts are organized under the state, county, city or similar unit of government, to perform one or more specific functions, as distinguished from the general function of governing. Special districts, for example, may be established for beach restoration or sand by-passing purposes, or for broader purposes of which beach restoration and sand by-passing are part. Existing special districts, such as port districts and oceanside road districts, originally organized for other purposes, may in some cases be found to have the authority to engage in beach restoration work. Special districts are usually established by state legislation. They are often authorized to issue bonds, and are usually authorized to tax within their boundaries. Their boundaries may be set to enable the principal beneficiaries of work to pay its cost. Special districts thus lend themselves particularly to the problem of financing beach restoration or maintenance in those cases where owners or communities along the beach are willing to pay, but the taxpayers in areas further removed from the coast are not.

The financial problems of special districts are the same as those of cities and counties in that no very great amount of money can ordinarily be raised at one time by direct taxation, that bond issues may therefore be required, and that the usual services must be engaged and the usual steps taken to successfully carry out a bond financing program.

ACKNOWLEDGMENTS

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MOTION OF SAND PARTICLES BETWEEN GROINS

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(Proc. Paper 1876)

SYNOPSIS

Experiments on the motion of sand particles between groins were performed in a fixed basin to compare with the results of groins in a movable bed. The comparison proved that both results were in comparatively good agreement.

INTRODUCTION

Arrangements of groins on a sandy beach were studied for two years, 1954 and 1955, in a basin with a movable bed, and the results were reported at the first and second Conferences of Coastal Engineering in Japan and in the Proceedings of the American Society of Civil Engineers, Separate Paper 1063, WW 4, September 1956. The experiments dealing with the motion of sand particles between groins were made in a basin with a fixed bed during the period April 1957 to March 1958. This paper presents a comparison of the results of the previous movable bed experiments with the results obtained in the fixed bed experiments.

Experimental Equipment and Procedure

Experimental Equipment

Experiments were performed in a basin built up with mortar, 11.40 m. long, 6.50 m. wide, and 0.40 m. deep as shown in Fig. 1. The beach was

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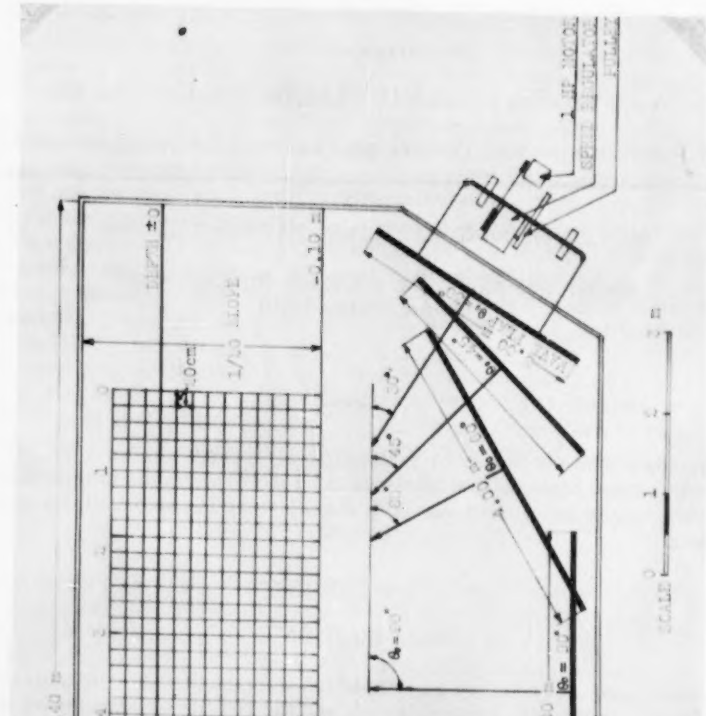

Fig. 1. Plan of the Experimental Basin

constructed to a 1/20 slope, and the offshore part of the toe of the slope was of constant 0.10 m. depth. The surface of the slope was painted black and 20 cm. x 20 cm. squares were formed with narrow white lines on the black slope, as shown in Fig. 1. The model groins, the same ones used previously in the movable bed experiments, were 50 cm. long, 2 cm. wide, 3 ~ 6 cm. high, and the length from the shoreline to the offshore end was 30 cm. The wave machine used was of the flap type and was portable in that it could be moved to produce waves at varying angles to the beach. The period and height of the waves produced could be varied by adjusting respectively the speed of the 1-HP motor and the length of the crank arms connected with the wave flap. The length of the wave flap also could be varied from 3 to 4 m.

Experimental Procedure

To measure the routes and velocities of motion of particles near the bottom, small particles made of trichloro-ethylene and toluene, and of $\gamma \doteqdot 1.05$ specific weight and of diameter $d \doteqdot 3$ or 4 mm. were used, while the motion of the surface particles were measured by particles made of parrafin wax, with $d \doteqdot 3 \sim 4$ mm. and $\gamma \doteqdot 0.83$.

To measure the motion of many particles at one time, a more ingenuous method was adopted. That was to take photographs at night by the use of fluorescent particles of $\gamma \doteqdot 1.06$ and 0.83 using fluorescent light.

The basin was filled with water to a depth of 0.1 m. and waves were propagated toward the beach at angles (Fig. 1) of 30, 45, 60, and 90 degrees. For each wave direction, a wave train of constant period (approximately 1.2 seconds) and a constant height (approximately 3.4 cm.) were used to test the direction and velocity of particles near the surface of the water and near the bottom from the breaking point of the shoreline. The same constant period wave from each direction was used to test the motion of the particles along a beach with no groins; along a beach with a single groin with its axis at angles of 60, 70, 80, 90, 100, 110, and 120 degrees to the updrift shoreline; and along a beach with four groins with their axes at angles of 70, 80, 90, 100, 110, and 120 degrees to the updrift shoreline. For the tests with four groins, groin spacings of 2ℓ, 3ℓ, and 4ℓ were used with most of the groin directions (ℓ equals the distance from the shoreline to the offshore end of the groin). Also, one test was made using a wave angle (θ_0) of 90 degrees and a groin angle (α) of 90 degrees in which the seaward ends of the groin were connected by a submerged barrier. The actual test conditions are summarized in Table 1.

Experimental Results

In the Case of $\theta_0 = 30^\circ$

1. On the Beach without a Groin.

The routes and approximate velocities of particles' motion at the water surface and bottom in this case are shown in Figs. 2(a) and (b), and Photograph 1. According to Fig. 2 and Photograph 1, after the waves break, particles near the surface approach the shoreline with a reduced angle, $\theta = 25^\circ$, because of the longshore current, while particles near the bottom move simultaneously offshore at an angle of approximately 20° to the shoreline.

Table 1--Kinds of Experiments

Offshore Wave					Breaking Wave			Setting of Groins		
Wave Direction θ_0	Period T	Wave Length L_0	Wave Height H_0	Steepness $\delta = H_0/L_0$	Breaking Point from the Shore-line	Wave Height H_b	Number	Space cm	Space Multiple of ℓ	Direction Angle to the Shoreline α
°	sec	cm	cm		cm	cm				°
30	1.21	111	3.5	0.032	60 ~ 70	4.0	0	--	--	
							1	--	--	60, 70, 80, 90, 100, 110, 120
							4	60	2	70, 80, 90, 100, 110, 120
							4	90	3	70, 80, 90, 100, 110, 120
							4	120	4	70, 80, 90, 100, 110, 120
45	1.20	110	3.3	0.030	65 ~ 70	4.0	0	--	--	
							1	--	--	90, 110
							4	60	2	90, 110
							4	90	3	90, 110
							4	120	4	90, 110
60	1.19	101	3.2	0.032	60	4.0	0	--	--	
							1	--	--	60, 70, 80, 90, 100, 110, 120
							4	60	2	70, 80, 90, 100, 110, 120
							4	90	3	70, 80, 90, 100, 110, 120
							4	120	4	70, 90, 100, 110, 120
90	1.20	115	3.4	0.028	60	4.6	0	--	--	
							1	--	--	90, 110
							4	60	2	90
							4	90	3	90
							4	120	4	90
							4	60	2	90
							4	90	3	90 with submerged structures
							4	120	4	90

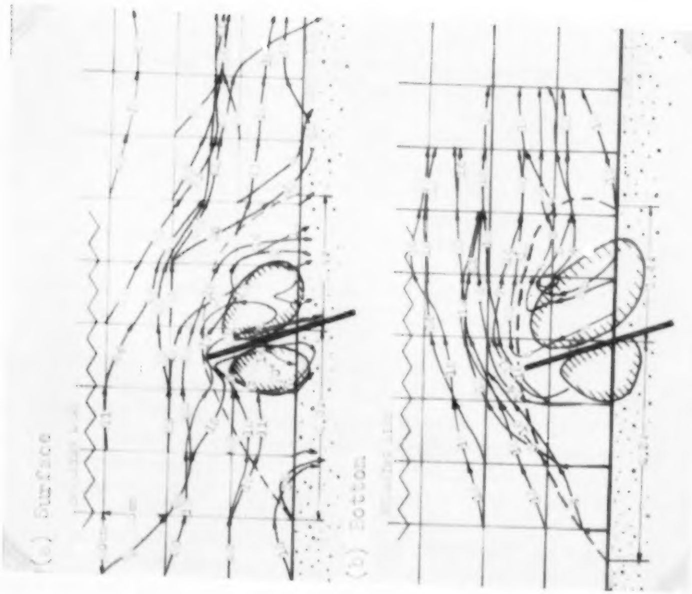

Fig. [illegible]. Routes and Velocities of Particles' Motion ($\theta_0 = 30°$, $\psi = 70°$)

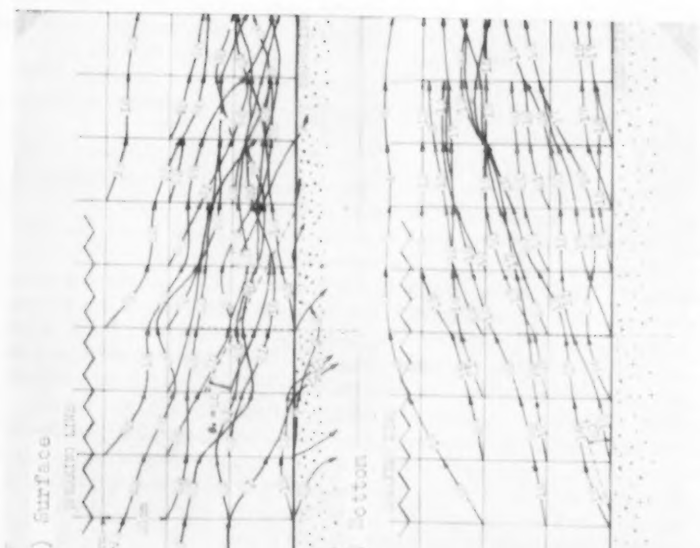

Fig. 4. Routes and Velocities of Particles' Motion ($\theta_0 = 30°$, No Groin)

2. On the Beach With One Groin.

When one groin was set at angles of $\alpha = 60^\circ$, 70°, 80°, 90°, 100°, 110°, and 120° to the shoreline, the routes and approximate velocities of particles at the bottom and surface around the groin were indicated as (a) and (b) of Figs, 3, 4, and 5 and Photographs 2, 3, and 4. (Figs. of $\alpha = 60^\circ$, 80°, 100°, and 120° are omitted.)

In Figs. 3 to 5, the boundary of the area where velocities were reduced or the particles direction of motion was changed by the groin is shown by broken lines. The areas between the boundary lines and the shoreline are shown in Table 2 with the ratio of these areas to the comparable area for $\alpha = 90^\circ$. Moreover, within these areas of influence there are regions where the particles' direction of motion is opposed to the direction of the longshore current or takes circular paths. These areas are shown by hatching in Figs. 3 to 5 and are referred to as "areas of shade" in Tables 2, 3, and 4. (Area in shadow of groin).

According to Figs. 3 to 5 and Table 2, the area of influence around a groin is a maximum when the groin is at an angle of $\alpha = 110^\circ$, as is the case of $\alpha = 90^\circ$. The area of shade is a maximum for angles of $\alpha = 90^\circ$ and 110°, with regard to the surface particles on the downdrift side of the groin.

It is obvious from Figs. 3 to 5 and Photographs 2 to 4 that some of the surface particles follow a circular path, while the bottom particles generally do not follow this motion but merely change their direction or decrease in velocity somewhat updrift. In these regions, at the updrift side of the groin, return currents are caused at the bottom. These currents tend to prevent the approach of waves containing sand, resulting in poor conditions for the deposition of drifting sand. Therefore, it may be considered that only the areas of influence and circulation at the surface are important factors in the effective deposition of drifting sand at the updrift side of a groin. On the other hand, it may be reasoned that the larger the areas of influence and shade with respect to the surface and bottom particles are at the downdrift side of a groin, the more drifting sands deposit at the downdrift side of a groin. This is true because in these areas the return current does not prevent waves containing sand from approaching.

Analyzing Figs. 3 to 5 and Table 2 in accordance with the previous discussion, the most effective direction of a groin in relation to sand deposition is $\alpha = 90^\circ$ and 110°. The groin is especially effective at the angle of $\alpha = 110^\circ$.

According to the experimental results in the movable bed basin, the root of the groins were scoured because waves strike against the groins along their updrift sides toward their roots, when $\theta_0 = 35^\circ$ and $\alpha = 90^\circ$. Under the same conditions there was not much deposition at the downdrift side of the groins because of the invasion by waves with high energy. On the other hand, when $\alpha = 110^\circ$, since waves striking the root along the updrift side of the groin were relatively weak, there was no scouring of the root. Rather sand was deposited near the root of the groin at the point where the waves breaking on the beach meet the backwash. Moreover, at the downdrift side of the groins, the diffracted waves of moderate energy resulted in a greater deposition of sand. These results can readily be verified by comparing Figs. 4 and 5, and therefore, it can be said that the experimental results in the fixed bed basin compare favorably with those in the movable bed basin.

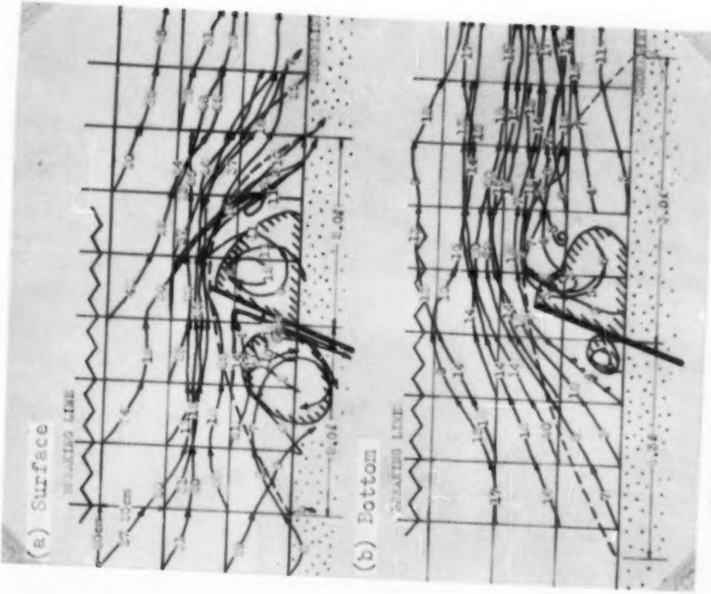

Fig. 5. Routes and Velocities of Particles' Motion ($\theta_0 = 30°$, $\alpha = 110°$)

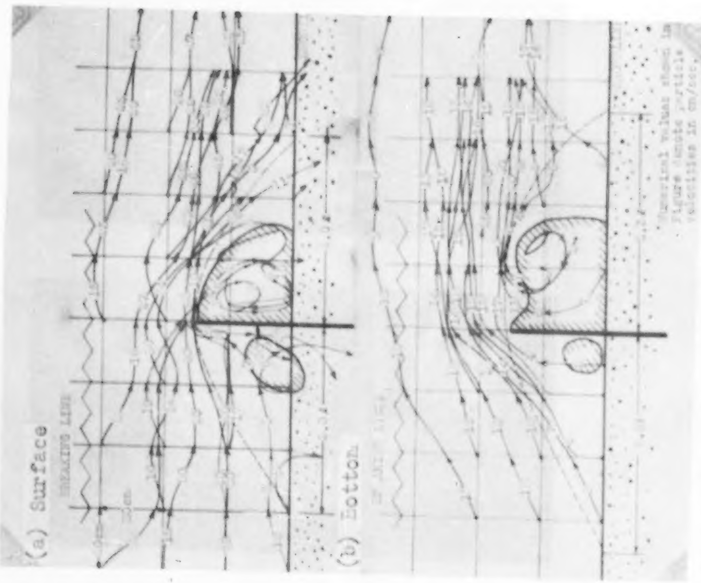

Fig. 4 Routes and Velocities of Particles' Motion ($\theta_0 = 30°$, $\alpha = 90°$)

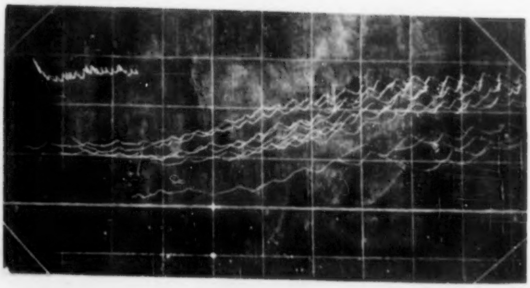

Photo. 1.
Routes of Particles' Motion at the Bottom
($\theta_0 = 30°$, No Groin)

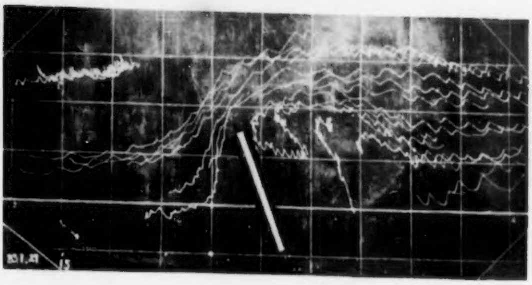

Photo. 2.
Routes of Particles' Motion at the Bottom
($\theta_0 = 30°$, $\alpha = 70°$)

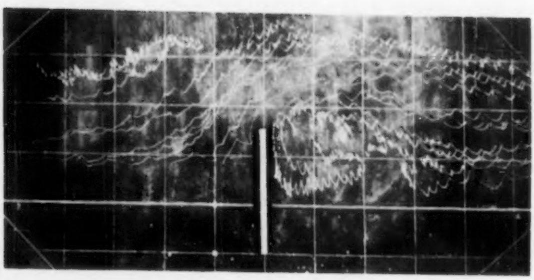

Photo. 3.
Routes of Particles' Motion at the Bottom
($\theta_0 = 30°$, $\alpha = 90°$)

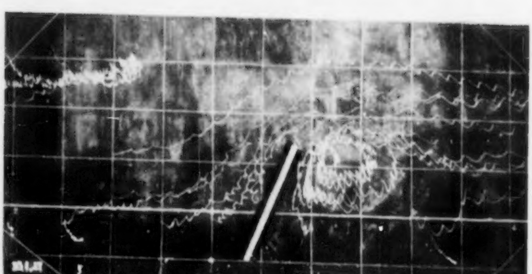

Photo. 4.
Routes of Particles' Motion at the Bottom
($\theta_0 = 30°$, $\alpha = 110°$)

Table 1.—Areas of Influence due to Setting of One Grain (θ = 90°)

Direction Angle of a Grain	Area of Influence												Area of Shade	
	Limit of Influence						Ratio of Area (to That of θ = 90°)						Ratio of Area	
	Surface Particles			Bottom Particles			Surface Particles			Bottom Particles			Surface Particles	Bottom Particles
	Up-Drift	Down-Drift	Sum	Up-Drift	Down-Drift	Sum	Up-Drift	Down-Drift	Mean	Up-Drift	Down-Drift	Mean	Down-Drift	
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Table 2.—Areas of Influence due to Setting of One Grain (θ = 90°)

Direction Angle of a Grain	Area of Influence												Area of Shade	
	Limit of Influence						Ratio of Area (to That of θ = 90°)						Ratio of Area	
	Surface Particles			Bottom Particles			Surface Particles			Bottom Particles			Surface Particles	Bottom Particles
	Up-Drift	Down-Drift	Sum	Up-Drift	Down-Drift	Sum	Up-Drift	Down-Drift	Mean	Up-Drift	Down-Drift	Mean	Down-Drift	
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Table 3.—Areas of Influence due to Setting of One Grain (θ = 90°)

Direction Angle of a Grain	Area of Influence												Area of Shade	
	Limit of Influence						Ratio of Area (to That of θ = 90°)						Ratio of Area	
	Surface Particles			Bottom Particles			Surface Particles			Bottom Particles			Surface Particles	Bottom Particles
	Up-Drift	Down-Drift	Sum	Up-Drift	Down-Drift	Sum	Up-Drift	Down-Drift	Mean	Up-Drift	Down-Drift	Mean	Down-Drift	
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3. On the Beach with Four Groins.

a. Space between Groins.

Judging from the results of experiments on the beach with one groin, it is considered that the space between groins must not be greater than that of the up- and downdrift stretches of beach influenced by one groin; that is, in the case of $\alpha = 110^{\circ}$ the spacing (D) between groins should be less than 5ℓ or 4ℓ as indicated by the paths of the surface and bottom particles.

The velocity of particles at distances of 0, $2/3\ell$ and $4/3\ell$ seaward from the shoreline and at points along the shoreline of $2/3\ell$, $4/3\ell$, and 2ℓ both updrift and downdrift from a groin are shown in (a), (b), and (c) of Figs. 6 and 7. From these figures the velocity reduction caused by a groin is seen to be quite large near the shoreline, even at distances of 2ℓ up and downdrift from the groin. However, at a distance of about $2/3\ell$ offshore from the shoreline, velocity reduction is very small at a distance of 2ℓ from a groin, and the groin has little influence on the particles' motion on the updrift side. It is, therefore, considered to be desirable to set the next groin downdrift so that its updrift area of influence to a distance of $2/3\ell$ normal to the shoreline is overlapped by the downdrift area of influence of the updrift groin. Applying this consideration to the bottom particles in the case of $\alpha = 90^{\circ}$ and 110°, the results are shown in Figs. 8(a) and (b). According to Figs. 8(a) and (b), the necessary distance (D) between groins is equal to 3.2ℓ for $\alpha = 110^{\circ}$ and 2.7ℓ for $\alpha = 90^{\circ}$.

The paths and velocities of the particles at the surface and bottom for groin spacings of $D = 2\ell$, 3ℓ, and 4ℓ are shown as (a), (b), and (c) in Figs. 9, 10, and 11, and Photographs 5 and 6 for the case of $\alpha = 70^{\circ}$, 90°, and 110°. (The other figures are omitted.) From these figures, the following trends are noted for all cases of $\alpha = 70^{\circ}$ to 120°.

1. There is a large circulation on the surface of water at the downdrift side of the first groin. This circulation grows larger as the space, D, between the first and the second groins increases, simultaneously the circulation moves gradually away from the first groin and moves to the middle of the space between the groins when the groin spacing, D, becomes 4ℓ. Under such conditions a sand bar is formed halfway between the two groins.
2. In the space between the second and the third groins, the surface particles move toward the shoreline between the 2.0ℓ to 2.5ℓ distance downdrift from the root of the second groin. Therefore, when $D = 2\ell$, waves strike upon the root of the third groin and cause scour thereby joining with the waves running along the third groin on its updrift side. When $D = 4\ell$, waves will strike upon the shoreline near the 2.5ℓ distance downdrift from the root of the updrift side and cause scouring. But when $D = 3\ell$, sand deposition can be expected on the updrift side of the root of the third groin because the approaching waves collide with a return flow near point 2.5ℓ downdrift from the second groin.
3. When $d = 2\ell$, it is thought that it is difficult for the surface particles to invade the spaces between groins.

From these results, the greatest deposition of drifting sand may be expected when the spacing of groins (D) is equal to 3ℓ. This result corresponds to the experimental result obtained in the movable bed basin.

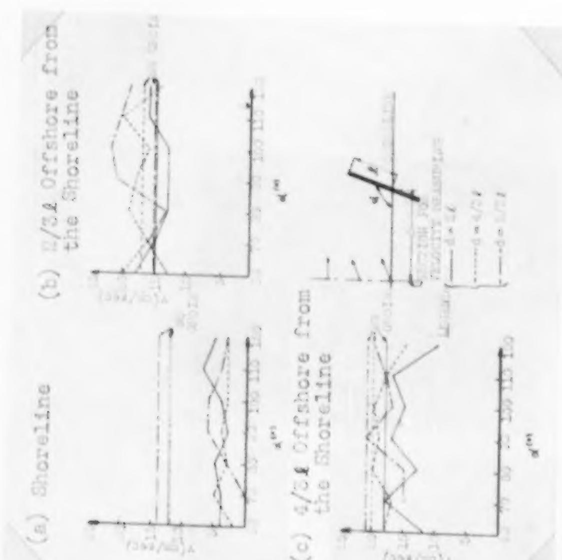


Fig. 6. Change of Bottom Velocities at the Up-Drift Side of One Groin

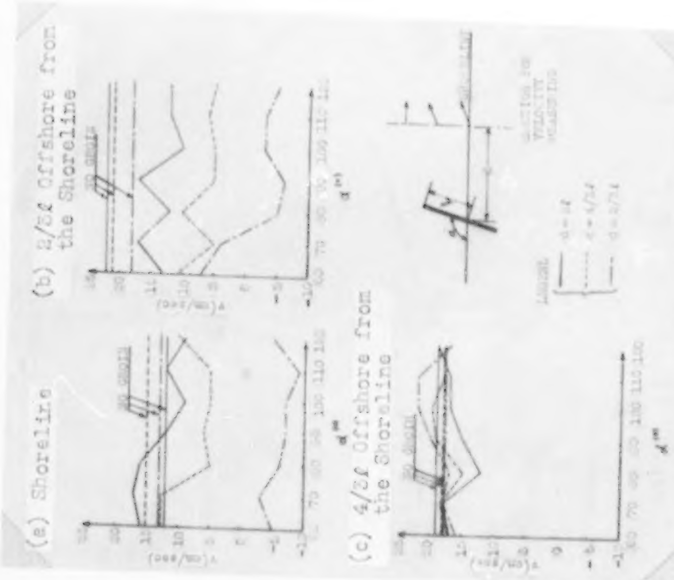


Fig. 7. Change of Bottom Velocities at the Down-Drift Side of One Groin

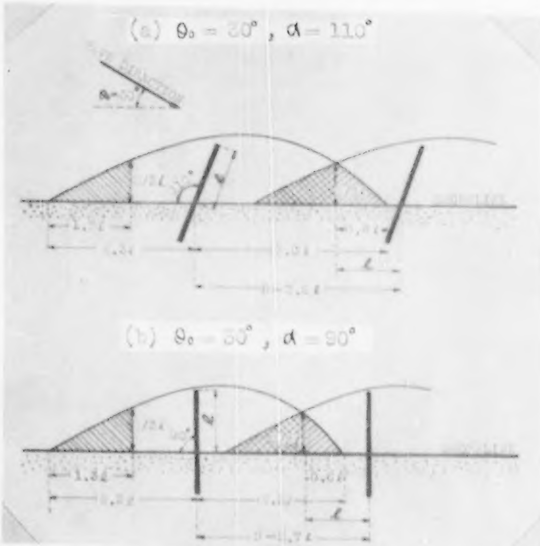


Fig. 8. Determination of the Space between Groins

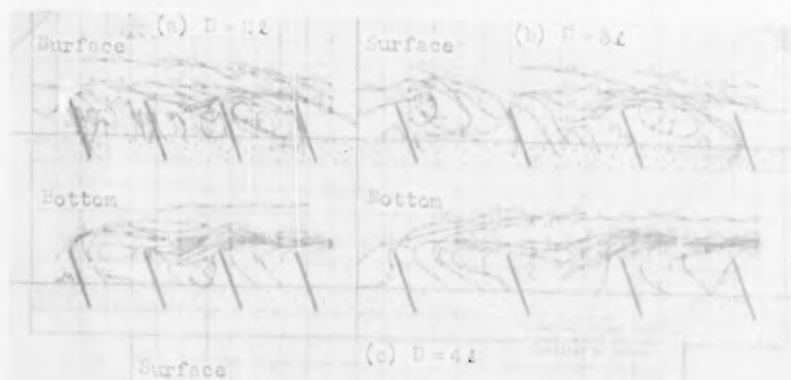


Fig. 9. Routes and Velocities of Particles' Motion ($\theta_0 = 30°$, $\alpha = 70°$)

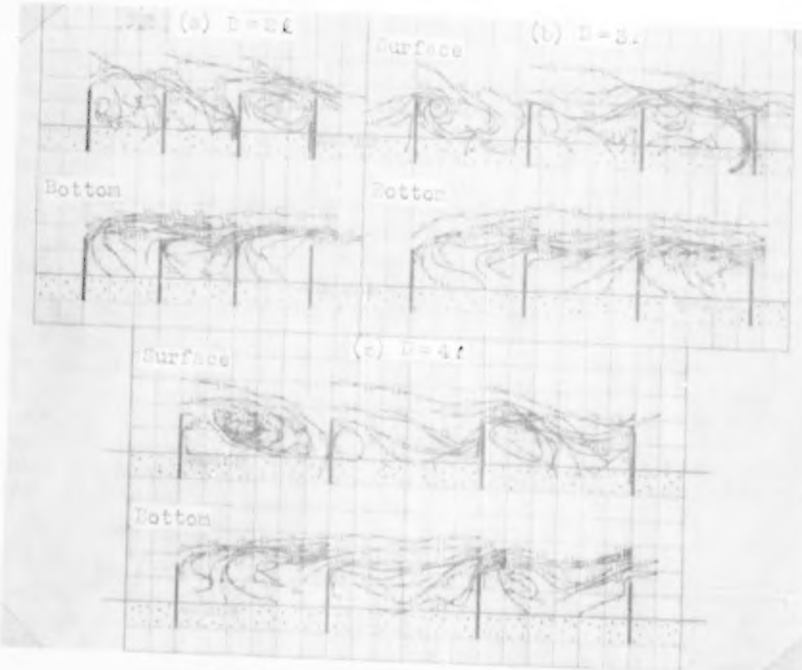
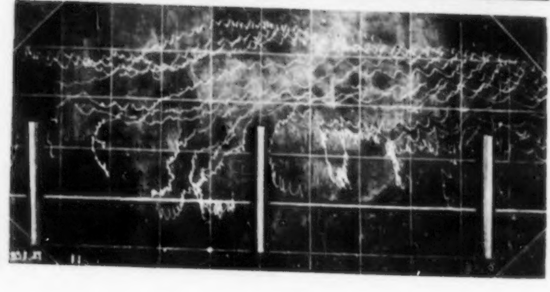


Fig. 10. Routes and Velocities of Particles' Motion ($\theta_0 = 20°$, $\alpha = 90°$)

Photo. 5. Routes of Particles' Motion at the Bottom ($\theta_0 = 20°$, $\alpha = 90°$)

(a) D = 2ℓ

(b) D = 3ℓ



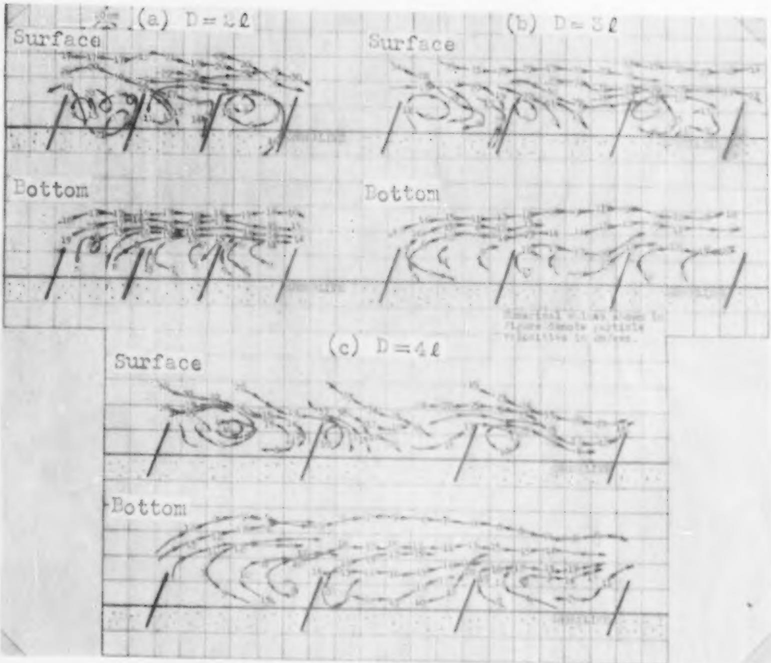
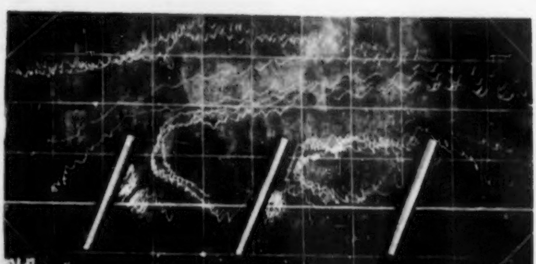


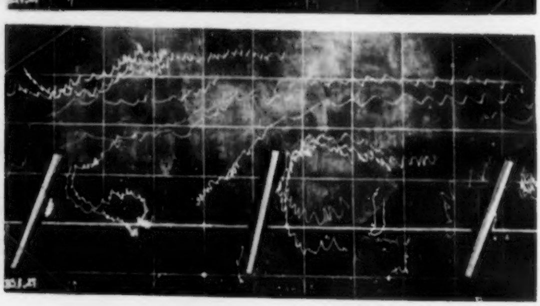
Fig. 11. Routes and Velocities of Particles' Motion ($\theta_0 = 30^\circ$, $\alpha = 110^\circ$)

Photo. 6. Routes of Particles' Motion at the Bottom ($\theta_0 = 30^\circ$, $\alpha = 110^\circ$)

(a) D = 2ℓ



(b) D = 3ℓ



b. Direction of Groins to the Shoreline

Though the experiments were carried out with angles varying from $\alpha = 70^\circ$ to 120°, and for groin spacings, D, equal to 2ℓ, 3ℓ, and 4ℓ, it was difficult to decide quantitatively which groin direction most effectively caused deposit of sand. However, it may be considered that when $\alpha < 90^\circ$ groins have a tendency to guide waves to their roots and when $\alpha = 120^\circ$ the updrift sides of the groins are liable to be scoured by the attack of waves with high energy.

According to the experimental results, the most effective direction of groins to the shoreline may be considered to be $\alpha = 110^\circ$.

In the Case of $\theta_0 = 60^\circ$

1. On the Beach Without a Groin.

The paths and velocities of particles near the water surface and bottom are shown in Fig. 12(a) and (b). According to Fig. 12, immediately inshore of the breaches particles near the surface move toward the shoreline at an angle of $\theta \doteqdot 45^\circ$, but the angle changes to $\theta \doteqdot 30^\circ$ to 32° at a distance of about 30 cm. (equal to the length of groins in water) offshore from the shoreline; while particles near the bottom run simultaneously offshore at an angle of 20° to 22°. This angle is as large as that for $\theta_0 = 30^\circ$.

2. On the Beach with One Groin.

When one groin was set at angles of $\alpha = 60^\circ$, 70°, 80°, 90°, 100°, 110°, and 120° to the shoreline, the paths and velocities of particles at the water surface and bottom around the groin are indicated as shown in (a) and (b) of Figs. 13, 14, and 15. Figures for $\alpha = 60^\circ$, 80°, 100°, and 120° are omitted. From these figures, the areas of influence and shade are shown in Table 3, with the ratio of these areas to that of $\alpha = 90^\circ$. According to Table 3, the limit of influence is the maximum at the angle of $\alpha = 110^\circ$ with regard to the surface and bottom particles, while the area of influence is the maximum at the angle of $\alpha = 90^\circ$. The area of shade at the downdrift side of the groin is, of course, the maximum at the angles of $\alpha = 100^\circ$ to 110°. The areas of shade at the updrift side of the groin cannot be compared because they are not clear. It is not desirable, however, to set one groin at angles of $\alpha = 100^\circ$ to 110°, because the root of the groin at the updrift side is scoured owing to wave action along the updrift side of the groin. Therefore, when $\alpha = 60^\circ$, it may be considered to be a little more effective for sand deposition to set a groin at an angle of $\alpha = 90^\circ$ than at $\alpha = 110^\circ$.

3. On the Beach with Four Groins.

a. Space between Groins.

The distribution of the particles' velocities at the points of 0, $2/3\ell$ and $4/3\ell$ offshore normal from the shoreline and at points along the shoreline of $2/3\ell$, $4/3\ell$, and 2ℓ both updrift and downdrift from a groin were measured by the same procedure as in the case of $\theta_0 = 30^\circ$. As a result, the effect due to setting of one groin was seen to be very small even at the points $2/3\ell$ updrift and downdrift from the groin at all angles of $\alpha = 60^\circ$ to 120°. It is, therefore, desirable to set the second groin so that the part $2/3\ell$ offshore from the shoreline in its updrift area of influence is overlapped with the downdrift area of influence of the updrift groin. Deciding the spacing between groins by

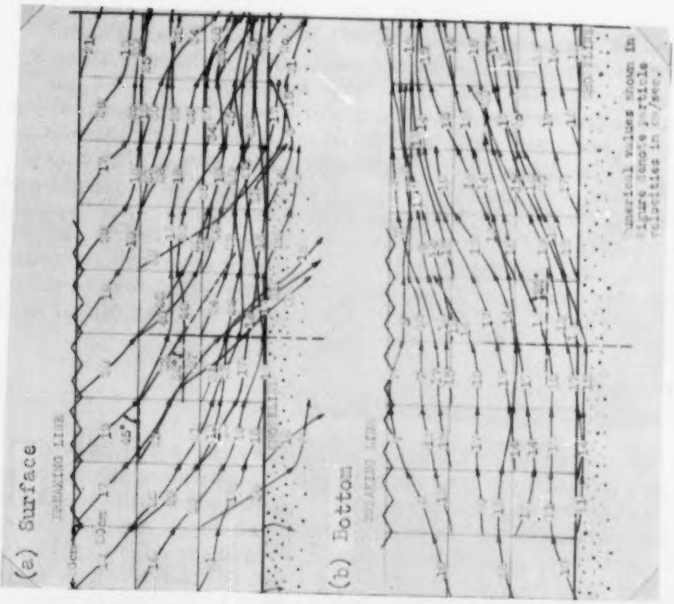


Fig. 12. Routes and Velocities of Particles' Motion ($\theta_0 = 60°$, No Groin)

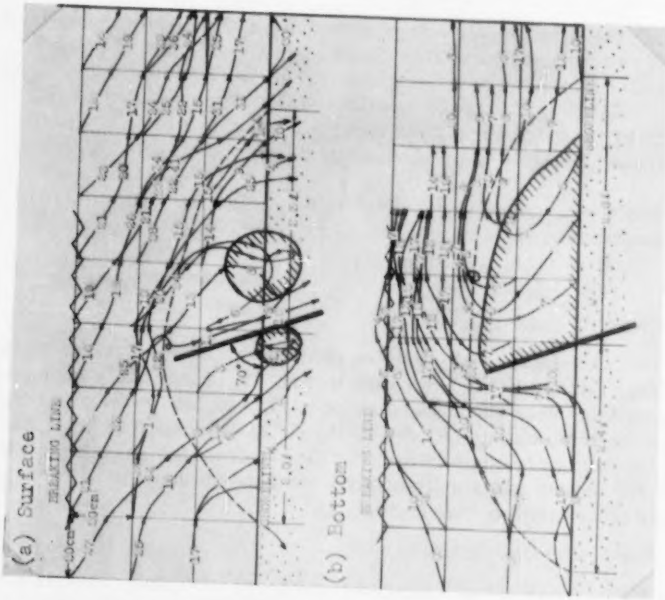


Fig. 13. Routes and Velocities of Particles' Motion ($\theta_0 = 60°$, $\alpha = 70°$)

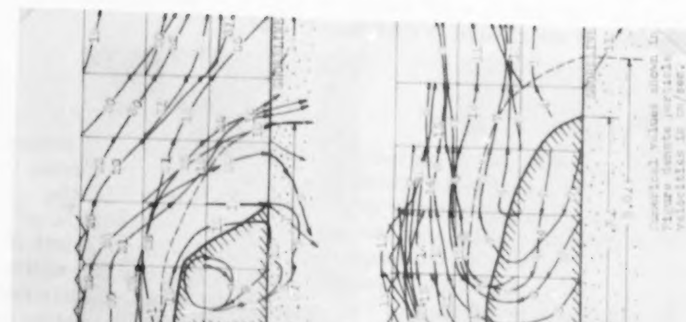


Fig. 14. Routes and Velocities of Particles' Motion ($\theta_0 = 60°$, $\alpha = 90°$)

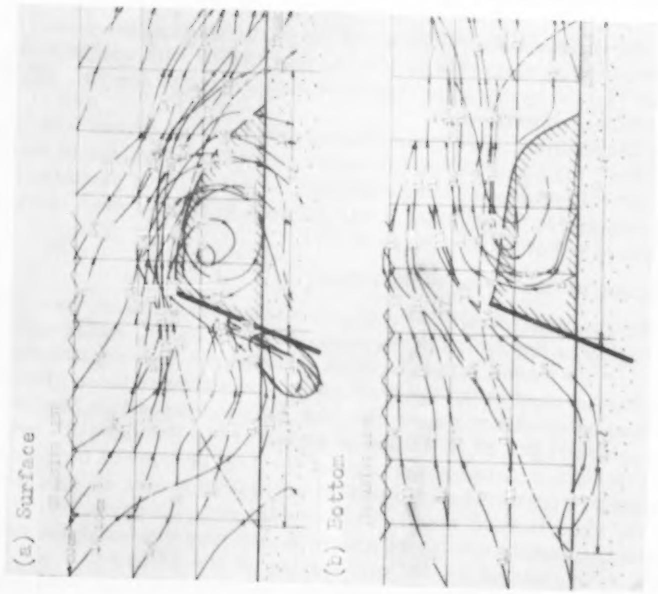


Fig. 15. Routes and Velocities of Particles' Motion ($\theta_0 = 60°$, $\alpha = 110°$)

this method, the relation $D = 3.9\ell$ is obtained in both the cases of $\alpha = 90^\circ$ and 110°.

The routes and approximate velocities of the particles near the water surface and bottom were measured by placing four groins with spacings of $D = 2\ell$, 3ℓ, and 4ℓ, and by changing their direction angles to $\alpha = 70^\circ$, 80°, 90°, 100°, 110°, and 120°, respectively.

The figures drawn from these measurements showed that groins with spacings of $D = 2\ell$ were most ineffective for sand deposition regardless of their angle with the shoreline because it was difficult for the surface particles to enter the spaces between the groins. It was difficult to decide which spacing of groins was more effective, $D = 3\ell$ or $D = 4\ell$.

b. Direction of Groins to the Shoreline.

As the direction of movement of the surface particles between groins is very close to that of the groins in the case of $\alpha = 70^\circ$ to 80°, waves with high energy scour the spaces between groins. Therefore, it is considered to be difficult to deposit sand in the spaces between groins under these circumstances. It would be considered most effective for sand deposition to place groins with spacings of $D = 3\ell$ to 4ℓ and at angles of $\alpha = 90^\circ$ and 110°, but the roots of the groins are scoured in the case of $\alpha = 110^\circ$ because of the concentration of the surface particles on the updrift side of the groins as described previously in the experimental results on the beach with one groin.

Judging from the results mentioned above, it may be considered desirable to place groins at an angle of $\alpha = 90^\circ$ with spacing of $D = 3\ell$ to 4ℓ.

The experimental results in the movable bed basin showed that sand deposition was maximum at an angle of $\alpha = 90^\circ$, and that it was more difficult to deposit sand at $\theta_0 = 60^\circ$ and $\theta_0 = 90^\circ$ than at $\theta_0 < 60^\circ$. A smaller amount of sand was deposited in the case of $D = 4\ell$ than $D = 3\ell$.

In the Case of $\theta_0 = 45^\circ$

1. On the Beach without a Groin.

The paths and velocities of particles at the water surface and bottom are shown in Figs. 16(a) and (b) and Photographs 7(a) and (b). According to Fig. 16 and Photograph 7, immediately after the breaking of waves, the surface particles are seen to advance toward the shoreline at an angle of about 45° and are not affected by the longshore current, but at the points 30 cm. offshore from the shoreline the value of θ reduces to about 25°, which is nearly equal to the value in the case of $\theta_0 = 30^\circ$. The bottom particules run simultaneously offshore at an angle of approximately 20° to the shoreline as in the case of $\theta_0 = 30^\circ$.

2. On the Beach with One Groin.

When one groin was set at angles of $\alpha = 90^\circ$ and 110° to the shoreline, the paths and velocities of particles near the surface and bottom were as indicated in (a), and (b) of Figs. 17 and 18 and Photographs 8 and 9. From these figures the areas of influence and shade due to setting of one groin are shown in Table 4, with the ratio of these areas to that of $\alpha = 90^\circ$.

According to Figs. 17, 18, and Table 4, the limit of influence on the shoreline in the case of $\alpha = 110^\circ$ is seen to be longer than that of $\alpha = 90^\circ$ with respect to both water surface and bottom particles, but the area of influence in the case of $\alpha = 90^\circ$ is a little larger than that of $\alpha = 110^\circ$ at both the updrift

(a) Surface

(b) Bottom

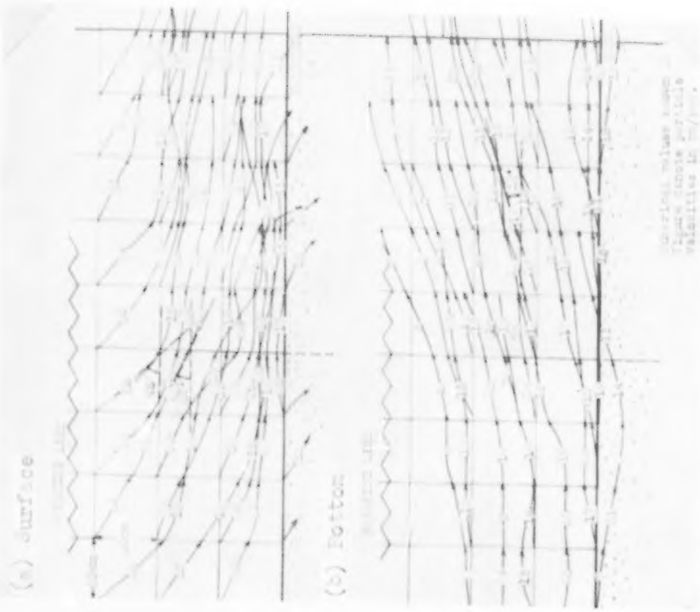


Fig. 16. Routes and Velocities of Particles' Motion ($\theta_0 = 45°$, No Groin)

(a) Surface

(b) Bottom

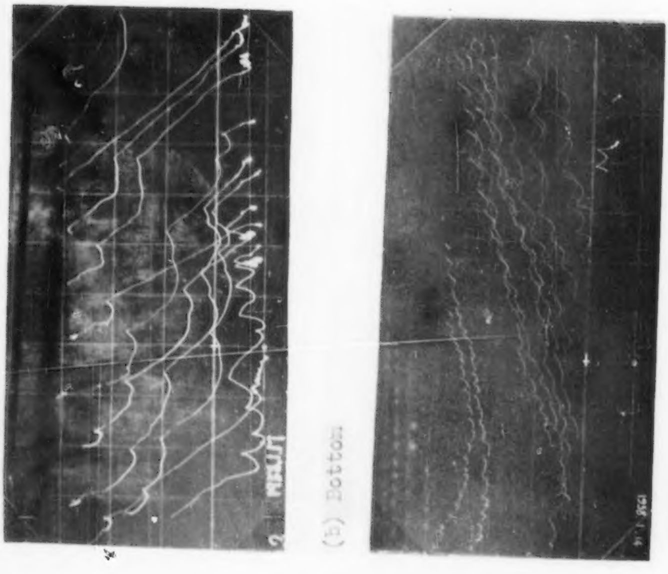


Photo. 7. Routes of Particles' Motion ($\theta_0 = 45°$, No Groin)

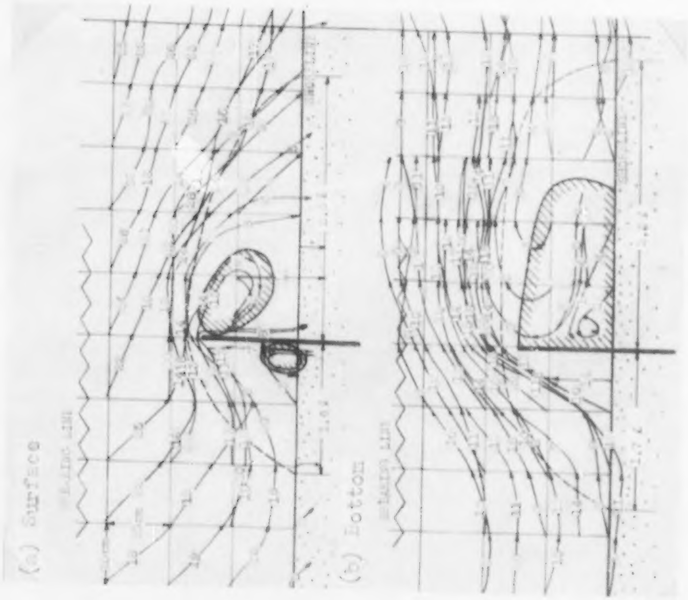


Fig. 17. Routes and Velocities of Particles' Motion ($\theta_0 = 45^\circ$, $\alpha = 30^\circ$)

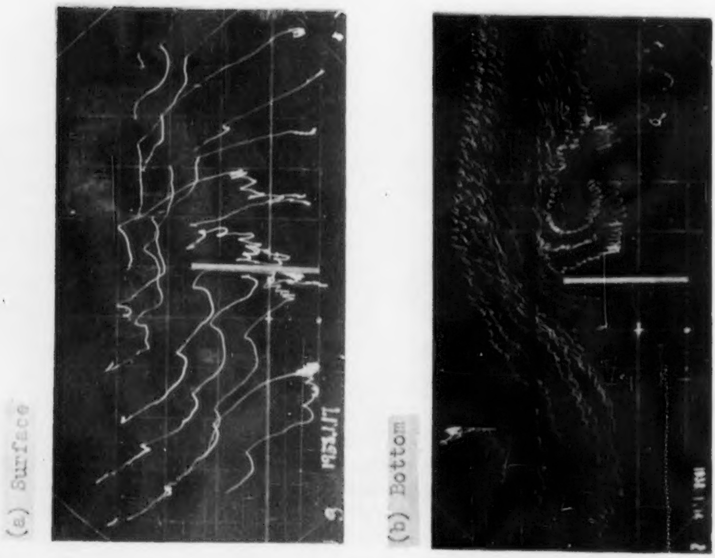


Photo. 8. Routes of Particles' Motion ($\theta_0 = 45^\circ$, $\alpha = 30^\circ$)

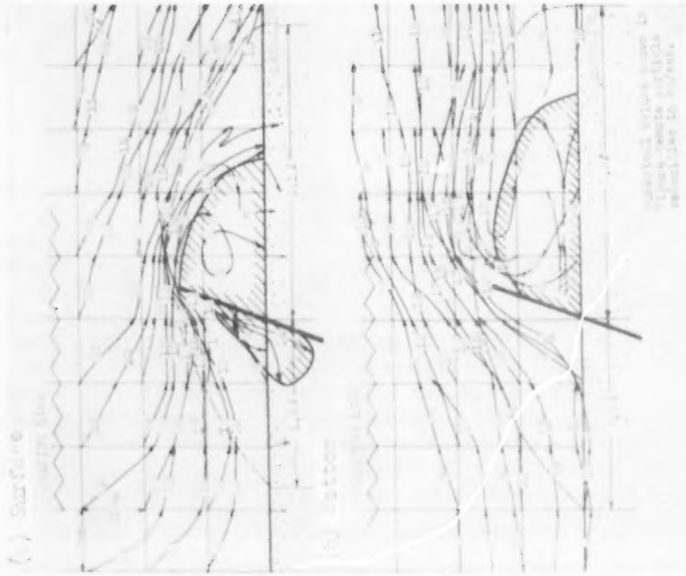


Fig. 13. Routes and Velocities of Particles' Motion (θ_0=45°, λ=110°)

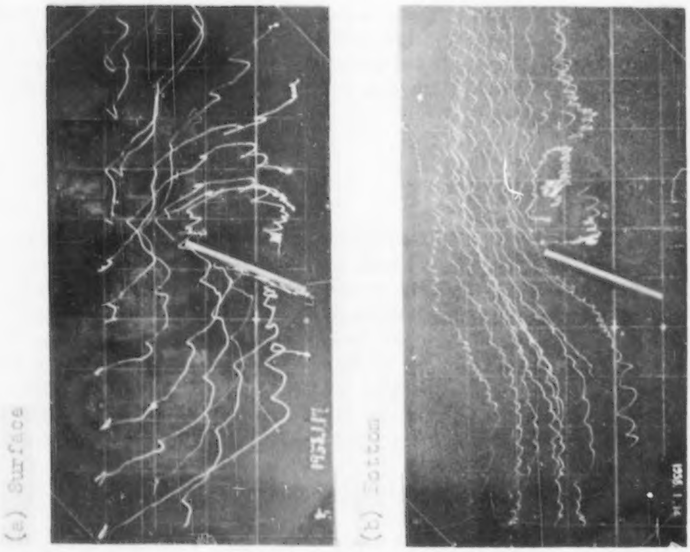


Photo. 9. Routes of Particles' Motion (θ_0=45°, λ=110°)

and downdrift sides. The area of shade at the downdrift side of a groin in the case of $\alpha = 110^0$ is seen to be larger than that of $\alpha = 90^0$ with respect to the surface and bottom particles. Therefore, it is difficult to decide which of the two directions would be more effective for sand deposition at the updrift and downdrift sides.

3. On the Beach with Four Groins

a. Space between Groins

Determining the spacing between groins by the same method as that in the case of $\theta_0 = 30^0$ and 60^0, D was equal to 4.1ℓ with respect to both $\alpha = 90^0$ and 110^0. The motion of particles near the water surface and bottom at angles of $\alpha = 70^0$ to 110^0 was measured, when $D = 2\ell$, 3ℓ, and 4ℓ (see (a), (b) of Photographs 10, 11, and 12).

According to the figures drawn by these measurements, it is shown to be difficult in the case of $D = 2\ell$ for the surface particles to enter the spaces between the groins, while in the case of $D = 4\ell$ it was considered that the beach would be scoured at about 2.5ℓ to 3ℓ distance downward from the updrift groin because the surface and bottom particles run parallel to each other offshore after striking against the shoreline at a point about 2.5ℓ to 3ℓ distant from the updrift groin. Therefore, it may be considered that the maximum sand deposition is caused in the case of $D = 3\ell$.

b. Direction of Groins to the Shoreline.

Considerable differences were not seen between $\alpha = 90^0$ and $\alpha = 110^0$, but it was considered to be a little more effective for sand deposition at the downdrift sides of groins to place the groins at an angle of $\alpha = 110^0$ than at $\alpha = 90^0$.

According to the experimental results in the movable bed basin, the maximum sand deposition resulted when the groin was at an angle of $\alpha = 110^0$ to the updrift shoreline and the space between groins was $D = 3\ell$. The experimental results in the fixed bed basin also showed the same tendency as the results mentioned above.

In the Case of $\theta_0 = 90^0$

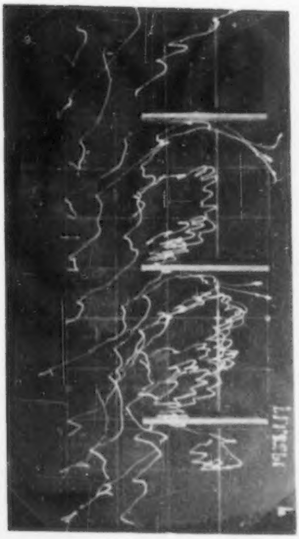
1. On the Beach without a Groin

When waves progress normally to the shoreline with the same wave height at any point along their wave crest, particles near the surface must reach the shoreline normally after the breaking of the wave, and particles near the bottom must run offshore simultaneously normal to the shoreline. In this experiment, however, particles ran from the center of the shoreline to both sides in small amounts as shown in Figs. 19(a) and (b), owing to the fact that the wave height of the central part of the beach was a little higher than that on both sides and that the wave crest lines were not perfectly parallel to the shoreline and the bottom of the fixed bed basin was not always perfectly plane.

2. On the Beach with One Groin.

When one groin was placed at an angle of $\alpha = 90^0$ to the shoreline, the routes and approximate velocities of particles near the water surface and bottom were as indicated in (a) and (b) of Fig. 20.

(a) Surface



(b) Bottom

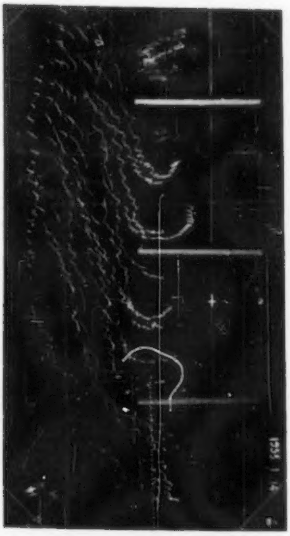
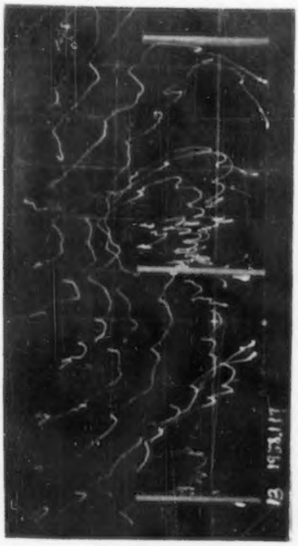


Photo. 10. Routes of Particles' Motion
($\theta_0 = 45^\circ$, $\alpha = 90^\circ$, $D = 2\ell$)

(a) Surface



(b) Bottom

Photo. 11. Routes of Particles' Motion
($\theta_0 = 45^\circ$, $\alpha = 90^\circ$, $D = 3\ell$)

(a) Surface

(b) Bottom

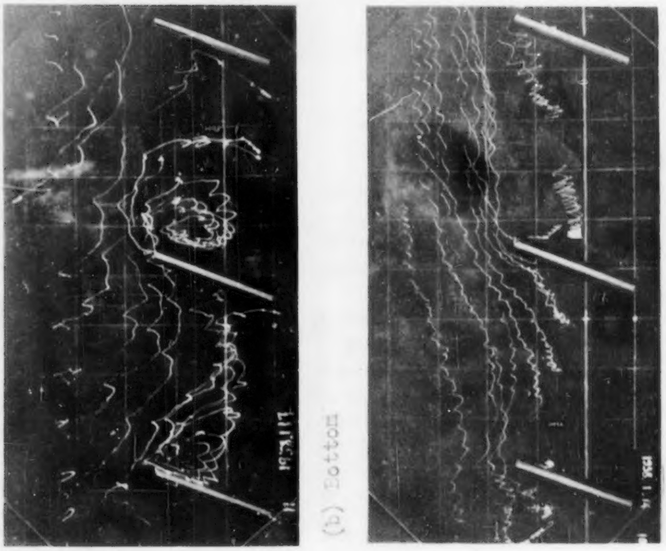


Photo. 11. Routes of Particles' Motion
($\theta_0 = 45^\circ$, $\alpha = 110^\circ$, $D = 3\ell$)

(a) Surface

(b) Bottom

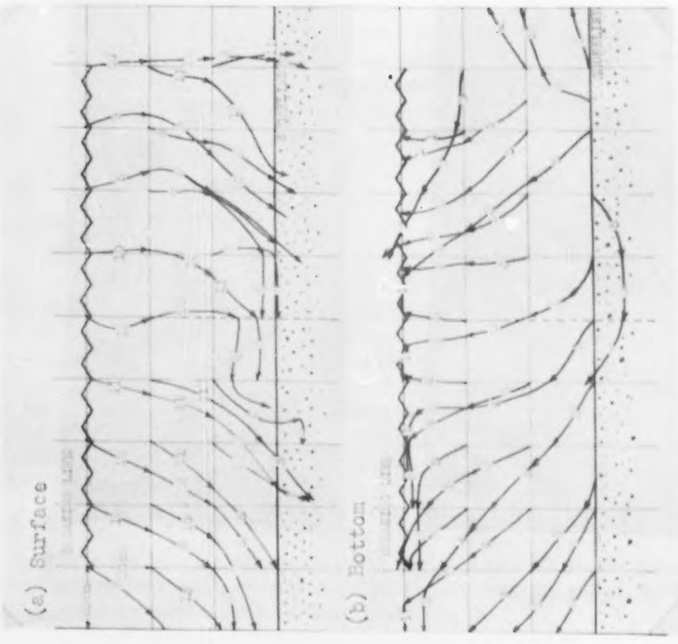


Fig. 15. Routes and Velocities of Particles' Motion
($\theta_0 = 30^\circ$, No Groin)

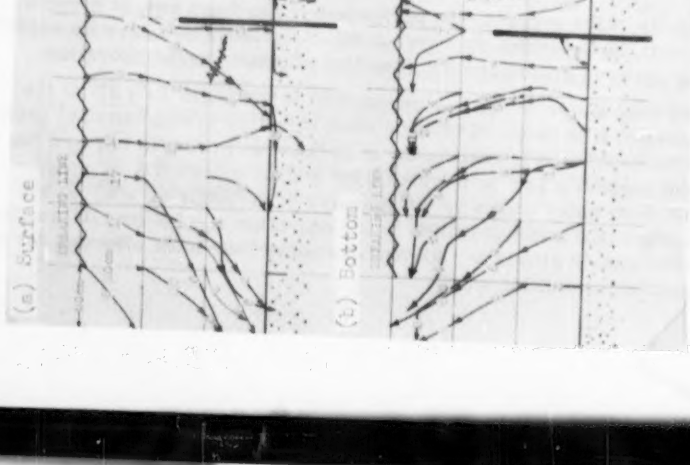


Fig. 20. Routes and Velocities of Particles' Motion ($\theta_0 = 90°$, $\alpha = 90°$)

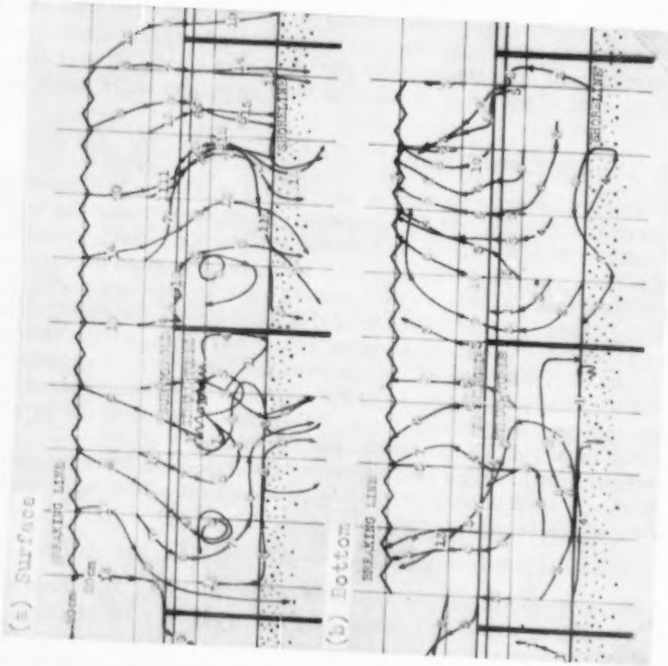


Fig. 21. Routes and Velocities of Particles' Motion ($\theta_0 = 90°$, $\alpha = 90°$, $D = 5\ell$, with Submerged Structures)

According to Fig. 20, at both the water surface and bottom, only particles near the groin moved approximately normal to the shoreline because of the effect of the groin, but their velocities were not changed appreciably. Velocities were changed only near the groin in the case of $\alpha = 110^\circ$. It was also obvious that the root of the groin at the side with an obtuse angle would be scoured.

3. On the Beach with Four Groins.

When four groins were located at an angle of $\alpha = 90^\circ$ to the shoreline with spacings of $D = 2\ell$, 3ℓ, and 4ℓ, respectively, particles near the water surface and bottom reached the shore and ran out approximately normal to the shoreline, and it was not expected that sand would be deposited due to these jetty-type groins alone. Also it was not possible for the surface particles to be deposited between groins in the case of $D = 2\ell$, because the rising of the water level on the shore produced a strong return current near the bottom. While in the case of $D = 3\ell$ and 4ℓ, small sand deposits would be possible because the rising of the water level near the shoreline was not produced and the return current was moderately weak. Also, the direction of the movement of particles had an inclination normal to the shoreline.

It would be very difficult, as was proven by the experiments in the movable bed basin, to expect sand deposition only by placing groins normal to the shoreline when waves strike normal to the shore.

4. On the Beach with Submerged Structures Parallel to the Shore.

The paths and velocities are indicated in (a) and (b) of Fig. 21 in the case where submerged structures connecting the offshore ends of groins were placed with their crowns a little lower than still water level, and Figs. 22 (a) and (b) show the case of $D = 3\ell$ without submerged structures in order to make a comparison with Fig. 21.

According to Fig. 21 in the case of $D = 3\ell$, the surface particles after passing over the submerged structures, decreased in velocity and fell into confusion; while the bottom particles decreased in velocity more appreciably than the surface particles and also fell into confusion. It was found that a large volume of sand deposited between the submerged structures and the shoreline because it was difficult for the particles passing over the submerged structures to run offshore over the structures again due to the return current.

This fact was also proved by the experiments in the movable bed basin.

CONCLUSIONS

The results of the experiments performed in the fixed bed, in general, coincided with those previously carried out in the movable bed, with regard to the spacing between groins and the direction of groins to the shoreline.

1. When deep water waves are propagated at the angle $\theta_0 = 30^\circ$ to the shoreline, it is believed that the most effective arrangement of groins for the deposition of sand in the area between groins is to place the axis at the angle $\alpha = 110^\circ$ to the shoreline and be spaced $D = 3\ell$ apart.
2. When deep water waves are propagated at the angle $\theta_0 = 45^\circ$ to the shoreline, the angle of groins to the shoreline $\alpha = 90^\circ$ or 110° and the space between groins $D = 3\ell$ are considered the most effective for the deposition of drifting sand.

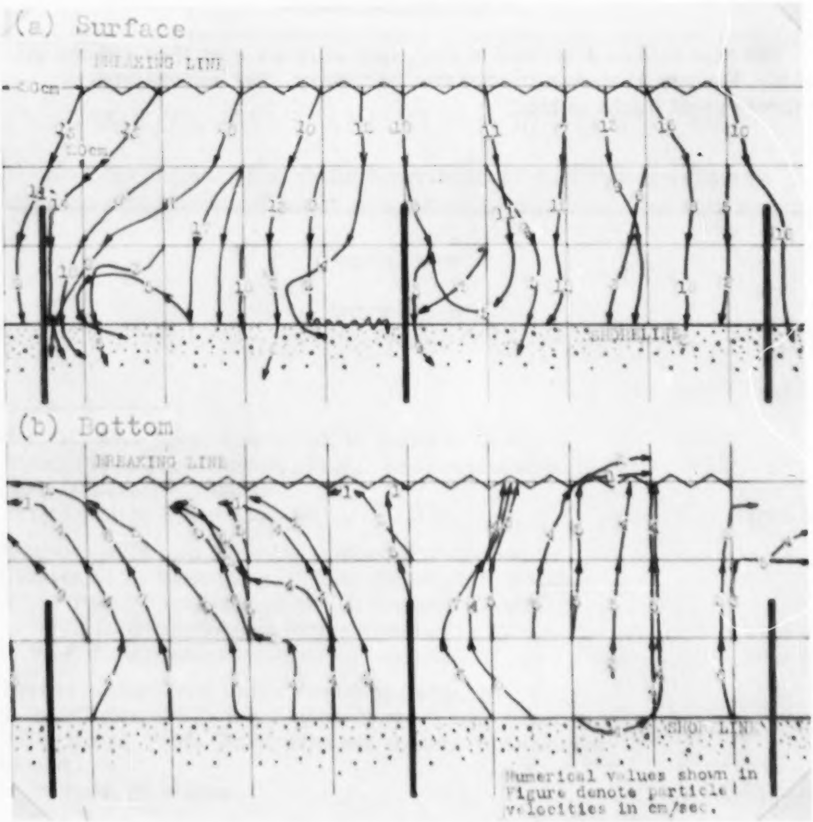


Fig. 22. Routes and Velocities of Particles' Motion
($\theta_0 = 90^\circ$, $\alpha = 90^\circ$, $D = 3\ell$)

3. When the direction of the propagation of deep water waves are $\theta_0 = 60^\circ$ and 90° to the shoreline, the angle of groins $\alpha = 90^\circ$ and the space between groins $D = 3\ell$ or 4ℓ are the most desirable to deposit drifting sands. When deep water waves invade nearly normal to the shoreline, it would be very difficult for a large amount of sand to be deposited in the area between groins only with the aid of jetty-type groins normal to the shoreline. It is more effective to place submerged structures parallel to the shoreline, connecting the offshore ends of the groins with their crowns a little lower than the still water level.

ACKNOWLEDGEMENT

The experiments described in this paper were accomplished with the aid of Mr. Kazuaki Akai, Assistant of this laboratory. The authors wish to express great thanks to him.

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WATERWAYS AND HARBORS DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION
(Proc. Paper 1884)

THE COLUMBIA RIVER CONTROLLED[a]

Closure by Louis H. Foote

LOUIS H. FOOTE,[1] M. ASCE.—This paper presents the significant considerations of water resource development planning in a major river basin with specific reference to the Columbia River Basin. Mr. Bessey submits a stimulating supplement to the original paper that evidences the deep interest and concern of informed individuals in adequate planning for the development and use of the water resources of the Columbia River Basin. The argument for full development of the Columbia River resource for maximum multiple use benefits is most convincing. However, the planner must evaluate all of the factors involved in the problem of Columbia River development, regardless of their nature, to determine what constitutes full development. Only by this process can an economically sound plan be evolved which will assure optimum tangible benefits. A plan based upon an exclusively theoretical study would, regardless of its excellence, be of only academic interest and value.

a. Proc. Paper 1514, January, 1958, by Louis H. Foote.
1. Brig. Gen., U. S. Army (Retired), Cons. Engr., Forest Grove, Ore.

SELECTION OF DESIGN WAVE FOR OFFSHORE STRUCTURES[a]

Corrections by C. L. Bretschneider[1]
Discussion by J. E. Chappelear

CORRECTIONS.— Page 1568-18 should be revised as follows:

water. The breaking wave height is given by, $H_b = L/7$, where L is the wave length, given by

$$L = 1.20 \frac{g}{2\pi} T^2 \tag{7}$$

The crest elevation above still water is obtained from $\eta_o/H = 0.676$. This theory is used for the upper limit in deep water, $d/L = 0.5$.

Stokes Theory (second approximation)(25)

This is a second order theory, one order above the Airy theory, and applies for waves of small finite steepness. The wave length is given by

$$L = \frac{g}{2\pi} T^2 \, (\tanh kd) \left[1 + \left(\frac{\pi H}{L}\right)^2 f_1 \left(\frac{d}{L}\right)\right], \text{ where}$$

$$f_1 \left(\frac{d}{L}\right) = \frac{5 + 2 \cosh 2kd + 2 (\cosh 2kd)^2}{8 (\sinh kd)^4} \tag{8}$$

The surface profile is given by

$$\eta/_H = \frac{1}{2}\left[\cos \theta + \frac{1}{2} \frac{\pi H}{L} f_2 (d/L) \cos 2\theta\right], \text{ where}$$

$$f_2 (d/L) = \frac{1}{2} \frac{(\cosh kd)(2 + \cosh 2kd)}{(\sinh kd)^3} \tag{9}$$

The elevation of crest η_o above still water is obtained from 9 for $\theta = 0$. This theory is used next to the boundary of the Airy theory.

Stokes Theory (third approximation for deep water)

The third approximation for deep water has a surface profile given by

$$\eta/_H = \cos \theta + \frac{\pi A}{L} \cos 2\theta + \frac{3}{2} \left(\frac{\pi A}{L}\right)^2 \cos 3\theta \quad \text{where} \tag{10}$$

A is related to $A_o = \frac{H}{2}$ according to Keulegan(26)

$$A_o = A \left[1 + \frac{3}{2} \left(\frac{\pi A}{L}\right)^2\right] \tag{11}$$

and the celerity is given by

$$C^2 = \frac{gL}{2\pi} \left[1 + \left(\frac{2 A}{L}\right)^2\right] \tag{12}$$

a. Proc. Paper 1568, March, 1958, by Charles L. Bretschneider.
1. Hydro. Engr., Beach Erosion Bd., Corps of Engrs., Washington, D. C.

This theory is used as the boundary for deep water, for waves of finite height, but is adjusted slightly as H/L approached 1/7.

Page 1568-20 Line 15, " . . . at still water, where $\theta = 0$," should read ". . . at still water, where $\eta = 0$."

Page 1568-21 Eq. (19) should read

$$\frac{\eta_{b/H}}{(L/L_A)^2} = \frac{g}{8\pi^2} \frac{(\tanh \frac{2\pi d}{L_A})^2}{H/T^2} \left[1 + (\frac{w_m}{C_b})^2\right]$$

Eq. (22) should read

$$\frac{u_{max}}{C} = 1 - \sqrt{1 - \frac{2g\eta_o}{C^2}}$$

Page 1568-22 Third line below TABLE I-2 (symbol $\emptyset$) should read (symbol ϕ)

J. E. CHAPPELEAR.[1]—It is surprising that most of the curves which are presented in this paper as extrapolation of limiting cases, i.e., deep and shallow water theories, supplemented by a considerable amount of empirical data, agree very well with as yet unpublished theoretical calculations of the properties of the highest wave which the writer has recently made, following the suggestion of Michell[2] in his original paper. The surprise is not that the experiments agree with the theory, but rather vice versa. The theoretical calculations were intended as a rough guide for computational purposes, and as a calculation aid for wave properties which are as yet unmeasured. Table 1 gives calculated numerical values corresponding to Figs. I-1, I-2, and I-3. If these points are plotted on the figures in the original paper, the agreement

Table I

H/L	d/L	y_b/T^2 ft/sec	d/T^2 ft/sec	H/T^2 ft/sec
.0579	.0666	.3521	.1990	.1729
.0636	.0775	.4297	.2524	.2074
.0696	.0885	.5144	.3111	.2444
.0755	.0996	.6038	.3735	.2834
.0921	.1328	.8918	.5822	.4038
.1059	.1659	1.191	.8098	.5171
.1167	.1988	1.484	1.0449	.6134
.1246	.2314	1.764	1.280	.6894
.1303	.2638	2.027	1.511	.7461
.1343	.2961	2.277	1.736	.7876
.1370	.3282	2.513	1.956	.8163
.1427	.6471	4.578	3.986	.8787
.1431	.9655	6.541	5.947	.8797

1. Shell Development Co., Houston, Texas.
2. Michell, J. H., On the Highest Waves in Water, Phil. Mag., 36, No. 5, 430-435 (1893).

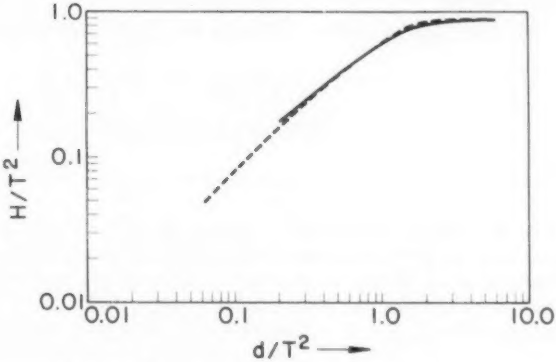

Fig. I-1 - Breaking Index Curve.

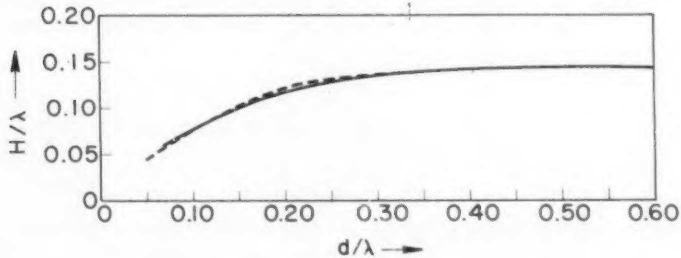

Fig. I-2 - H/λ Versus d/λ For Maximum Waves.

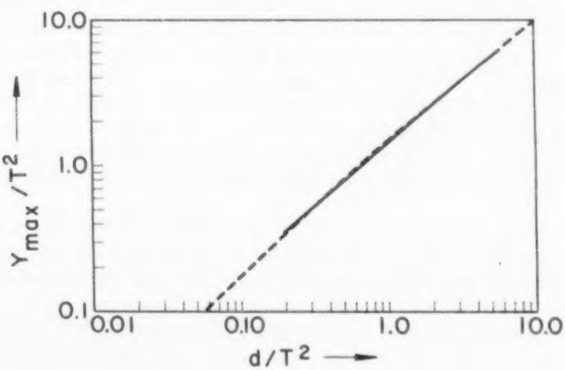

Fig. I-3 - Breaking Wave Crest Elevation.

is rather interesting. The notation is defined as follows: Notation d = mean water depth (ft); H = wave height (ft); L = wave length (ft); T = wave period (sec); and y_b = distance from crest to bottom (ft).

FORCES INDUCED ON A LARGE VESSEL BY SURGE[a]

Discussion by Basil W. Wilson

BASIL W. WILSON,[1,2] A. M. ASCE.—The author's paper is of considerable interest and value as a contribution to the literature on ship mooring forces and the more general problem of surging in harbors. In referring to the writer's paper (1951), the authors have recorded that the essential result for the period of longitudinal oscillation in surge of a moored ship, as obtained therein, is expressible in the form

$$T_n = 2\pi \left(\frac{m}{k}\right)^{\frac{1}{2}} \left(\frac{m g A^2}{k d}\right)^{j/2} \tag{7}$$

where $k = DN/4$ and $j = (1 - n)/(1 + n)$. It seems to the writer that this interesting analogy to the simple statement for an undamped, linear oscillating system, could perhaps have been more easily drawn by writing

$$T_n = 2\pi \left(\frac{m}{k'}\right)^{1/2} \tag{8}$$

where k' is a non-linear spring factor given by

$$k' = \frac{ND}{4} \left(\frac{NDd}{4mgA^2}\right)^j \tag{9}$$

The authors have further stated that the writer's result for the maximum total force in the moorings (R_{max}) reduces to

$$R_{max} = \frac{2\pi W A}{T_n \sqrt{gd}} \tag{10}$$

which Russell (1957) explains as exactly equivalent to the maximum force in the cables if the latter are tight, the ship held stationary and the virtual mass of the ship neglected. Actually from Eqs. (34a) and (25) of the writer's paper, it will be found that the writer's result was just twice that of Eq. (10) above while Russell's statement, as given by him, is in error by a factor $(g/2\pi)$.

The authors have rightly pointed to the discrepancies in this field that have come to light in their studies of the literature. Russell's result, they find, for the case when a ship is artificially released from rest at the moment that the water particles at the node of the seiche are also at rest, reduces to

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a. Proc. Paper 1571, March, 1958, by J. T. O'Brien and D. I. Kuchenreuther.
1. Contribution from the Dept. of Oceanography and Meteorology, Agricultural and Mechanical College of Texas, Oceanography and Meteorology Series No. 123.
2. Associate Prof. (Engineering Oceanography), A. & M. College of Texas, College Station, Texas.

precisely 1.5 times that given in Eq. (10). Russell gives no explanation for his derivation of this factor though it must be presumed that it is simply a mass factor, C_M (using the author's terminology). The writer's result, on the other hand, is equivalent to Eq. (10) multiplied by a C_M value of 2.0, which is probably too high, as will later be explained.

As the authors state, Joosting (who assisted the writer in South Africa on the model studies of surging in Table Bay Harbor at Cape Town), has recently (1957) pointed up an omission in the writer's original theory in respect of the influence of the wave slope, but has retained the writer's concept of an acceleration force due to the relative acceleration of the water moving past the ship. This, they note, is considered by Russell (1957) to be in error.

There is obvious need for clarification on these somewhat divergent views, and as their own contribution towards framing the problem analytically, the authors have advanced Eq. (1), the terms of which are covered by succeeding Eqs. (2) to (6). Eq. (1), however, would seem to be valid only so long as the inertial term, Eq. (2), is taken negative. The writer accepts the statements of Eq. (6) for the excitation force, expressive of net fluid pressure acting on the ship, provided that L (not defined) is the wave length and not the basin length. It can, in point of fact, be quite readily proved that the force from the net horizontal standing-wave pressures acting on a circular cylinder floating in water with its axis lying in the free surface along the node is exactly equivalent to the tendency of the cylinder to slide up or down the water slope. In respect of Eq. (2), however, the writer considers the treatment rather too summary and would wish to return to the point, refuted by Russell, that consideration is necessary of the time-rate-of-change of the velocity of the water relative to the ship.

The equation of motion of a cylinder of mass m accelerated by a force F in the positive x direction through an inviscid fluid initially at rest has been shown by Lamb (1932 Edn.) and others, to be

$$m\ddot{x} = F - m''\ddot{x} \tag{11}$$

where m'' is the so-called added mass and the term $m''\ddot{x}$ represents the resistance offered by the fluid pressures in the resulting potential flow, in which, relatively, the fluid accelerates backward past the cylinder at the rate $\ddot{x}$. If now the cylinder be drawn through a fluid which is flowing with varying velocity, u(t), in the same direction of pull, the relative velocity backward of the water past the cylinder is $(\dot{x} - u)$, and the resistance from its acceleration is $m'' \, d/dt(\dot{x} - u)$. The equation of motion thus becomes
$m\ddot{x} = F_1 - m'' \, d/dt\,(\dot{x} - u)$ or

$$m'\ddot{x} = F_1 + m''\dot{u} \tag{12}$$

where m' $(= m + m'')$ is the virtual mass of the cylinder. Obviously, if the water accelerates equally with the cylinder, there can be no net fluid pressures on the cylinder opposing the motion and the only force required to pull the body is that required to overcome the inertia of the body namely, $m\ddot{x} = F_2$.

In the case, now, of a floating cylinder, such as a ship, subject to wave action, the force F_1, propelling the ship, is provided by the net pressures in the direction of motion (deriving from the acceleration of the fluid) evaluated over the boundary of the cylinder, as if the cylinder were not present. In other words, the force on any imaginary boundary already exists in an

accelerating fluid regardless of the presence of any such real boundary. This is the excitation force referred to by the authors as the integrated components of the wave-induced pressure over the wetted surface of the ship.

Accordingly, the equation of motion of a ship moving across the node of a standing wave under the stimulus of the accelerative flow is given by Eq. (12) by suitably replacing F_1 in terms of the author's Eq. (6b) and by introducing the appropriate value of $\dot{u}$. If in addition the damping effect of the friction and form drag of the relative flow be considered and the restraints of mooring ropes be introduced, Eq. (12) becomes

$$m'\ddot{x} = mg(KA)\sin\sigma t + m''\dot{u} - \frac{1}{2}C_d\,\rho A'(\dot{x} - u)^2 - \sum_{i=1}^{N/2} k_i x^{n_i} \tag{13}$$

where $K = 2\pi/L$, $\sigma = 2\pi/T$ and A' refers to the projected (or wetted) area of the ship. On the assumption that the mass of the ship is entirely concentrated at the node of the seiche or standing wave, $\eta = A\cos Kx'\sin\sigma t$, where x' is taken with reference to some origin for the standing wave system at still water level, the values of u and $\dot{u}$ at $\dot{x}' = L/4$ (the node point) are given by

$$\left.\begin{aligned} &\text{(i)} \quad u = -A\sqrt{\frac{g}{d}}\cos\sigma t \\ &\text{(ii)} \quad \dot{u} = A\sigma\sqrt{\frac{g}{d}}\sin\sigma t\ . \end{aligned}\right\} \tag{14}$$

The friction term, which is small (as shown by the authors), may be linearized to the form $G(\dot{x} - u)$ with G a constant and Eq. (13) can then be rewritten with the aid of Eq. (14):

$$m'\ddot{x} + G\dot{x} + \sum_{i=1}^{N/2} k_i x^{n_i} = mgKA\sin\sigma t + m''A\sigma\sqrt{\frac{g}{d}}\sin\sigma t - GA\sqrt{\frac{g}{d}}\cos\sigma t\ . \tag{15}$$

Noting that m' is equivalent to the author's C_M m, Eq. (15) is in accord with the author's concept but contains two additional terms in the forcing function on the right hand side. On dividing through by C_M m and gathering terms, Eq. (15) reduces to

$$\ddot{x} + \frac{G}{C_M m}\dot{x} + \frac{1}{C_M m}\sum_{i=1}^{N/2} k_i x^{n_i} = \frac{A}{C_M}\left[gK + C_m\sigma\sqrt{\frac{g}{d}}\right]\sin\sigma t - \frac{GA}{C_M m}\sqrt{\frac{g}{d}}\cos\sigma t \tag{16}$$

If it be assumed that the various ropes i can collectively be considered to impose a restraint Cx^n, Eq. (16) may be expressed, after further simplification, as

$$\ddot{x} + \left(\frac{G}{C_M m}\right)\dot{x} + \left(\frac{C}{C_M m}\right)x^n = \sigma A\sqrt{\frac{g}{d}}\left(\frac{1 + C_m}{C_M}\right)\sin\sigma t - \frac{GA}{C_M m}\sqrt{\frac{g}{d}}\cos\sigma t\ . \tag{17}$$

This, substantially, is the equation given by Joosting (1957) for a C_M value of 2. The writer's original differential equation (Eq. (20) in his paper of 1951) was of this form (with $C_M = 2$) but had reversed (incorrect) signs to the forcing function on the right hand side and underestimated the first term of the latter by $1/C_M$. At that time, the writer was unable to find a formal solution to his equation but managed to evade the obstacle by recourse to dimensional analysis involving the quantities $(G/C_M m)$, $(C/C_M m)$, σ and $(A\sqrt{g/d})$, with $C_M = 2$.

It will be pertinent here to review the solution obtained by this method but for an unspecified value of C_M. To simplify matters we write

$$\left.\begin{aligned} &\text{(i)} \quad G/C_M m = 2\beta\omega \\ &\text{(ii)} \quad C/C_M m = \omega^2 \\ &\text{(iii)} \quad A\sqrt{g/d} = V \end{aligned}\right\} \tag{18}$$

so that Eq. (17) becomes, with $C_M = 1 + C_m$,

$$\ddot{x} + 2\beta\omega\dot{x} + \omega^2 x^n = V(\sigma\sin\sigma t - 2\beta\omega\cos\sigma t)\ . \tag{19}$$

The formal solution of Eq. (19) for n = 1 is easily obtained and yields

$$x = e^{-\beta\omega t}(a\sin\gamma t + b\cos\gamma t) + f(t)$$

where a and b are constants, f(t) is the forced oscillation and $\gamma = \omega(1 - \beta^2)^{1/2}$. The natural period of the free oscillation, T_1, is given by $T_1 = 2\pi/\gamma$ or

$$T_1 = \frac{2\pi}{\omega}[1 - \beta^2]^{-1/2} \tag{20}$$

By dimensional analysis it can be shown that the corresponding natural period in the general case will be

$$T_n = (\omega^2)^{-i}V^j\,\phi[(2\beta\omega)(\omega^2)^{-i}\cdot V^j] \tag{21}$$

where

$$\left.\begin{aligned} &\text{(i)} \quad i = \frac{1}{1+n} \\ &\text{(ii)} \quad j = \frac{1-n}{1+n} \end{aligned}\right\} \tag{22}$$

For the case of n = 1, Eq. (21) gives

$$T_1 = \frac{1}{\omega} \phi(\beta)$$

which agrees identically with Eq. (20) if

$$\phi(\beta) = 2\pi \left[1 - \beta^2\right]^{-1/2} \tag{23}$$

The writer assumed (1951) that the function ϕ in Eq. (21) could simply be approximated by 2π, thus yielding the result

$$T_n = 2\pi \, (C/C_M m)^{-i} \, (A \sqrt{g/d}\,)^{j} \tag{24}$$

This is identical to Eqs. (7) or (8) if

$$\left.\begin{array}{ll} \text{(i)} & k = DN/2C_M \\ \text{(ii)} & k' = \dfrac{ND}{2C_M} \cdot \left(\dfrac{NDd}{2C_M mgA^2}\right)^{j} \end{array}\right\} \tag{25}$$

It thus transpires that the writer's 1951 results, despite the error in the forcing function of the differential equation, are quite valid for the case of $C_M = 2$ within the limits of the assumption regarding $\phi = 2\pi$.

Abramson and the writer (1955) finally gave the mathematical solution of the writer's original differential equation. The error in the forcing function was unfortunately inherited but the method remains unaffected and the results are correct in all but their quantitative aspects. It was there shown that the writer's 1951 result (Eq. (24)) for the case of resonance for which $T_n = T$ (the wave period), could be expressed in the form

$$\left(\frac{\sigma}{\omega}\right)^2 = X^{n-1} \tag{26}$$

where X represents the maximum amplitude of the ship movement. The mathematical solution, however, yielded for the free, undamped oscillation

$$\left(\frac{\sigma}{\omega}\right)^2 = X^{n-1} \psi(n) \ , \tag{27}$$

where $\psi(n)$ is a numerical function of n only, from which it was concluded that a selection of $\phi = 5/2\pi$ in Eq. (21), rather than $\phi = 2\pi$, would have been a closer approximation to the truth.

For the forced oscillation without damping the solution was

$$X^n \psi(n) - \left(\frac{\sigma}{\omega}\right)^2 X = -\frac{V}{\omega} \tag{28}$$

and the maximum restoring force therefore

$$R_{max} = CX^n \tag{29}$$

Joosting (1957) has independently solved the nonlinear Eq. (19). His Eq. (10) corresponds identically with $1/\psi(n)$, above mentioned. Apart from his retention of a C_M value of 2, his results can be considered as fairly representative of the surging motion of a moored ship and of the restraint forces developing in the mooring lines.

The authors have indicated from their findings that C_M varied from about 1.0 to 1.2 for the "Norton Sound" in surge motion. Such a value can be confirmed from the formulae given by Wendel (1956). An approximate value of the hydrodynamic mass, m", in terms of the fluid density ρ, the beam B and draft D of the ship is

$$m'' = 0.26 \pi \rho B^2 D \;, \tag{30}$$

which for the quoted dimensions of the ship, yields $C_M = 1.20$. It would thus seem that the value of 2 used by the writer and by Joosting is much too high. If allowance be made for this in the writer's original equation for the maximum total force in the moorings, its expression in the form of Eq. (10) would be

$$R_{max} = \frac{2\pi C_M W A}{T_n \sqrt{gd}} \;, \tag{31}$$

which would be akin to Russel's result (for $C_M = 1.5$). Eq. (31), however, must now be considered superseded by Eqs. (28) and (29).

The condition of the mooring lines of the "Norton Sound", as remarked by the authors, accords with the linear case of n = 1, for which according to both Eqs. (26) and (27), resonance would only obtain with $\sigma = \omega$, the forced period of the wave agreeing with the free period of ship oscillation. The free period was found to be of the order of 30 seconds but no equivalent period surge-waves could be detected. The authors thus rightly concluded that the ship motion was non-resonant, as, indeed, may be inferred also from the fact that the motion was in phase with the excitation, for which σ/ω would have to be less than 1.0.

In Fig. 10 the authors detect only the presence of 0.98 and 2.50 min. waves but between the period 1104 and 1110 there seems to be clear evidence of a 0.76 min. oscillation. In Fig. 9, too, the rope tensions at the corresponding interval suggest an excitation of this order. With a degree more of slackness in the mooring ropes it would seem that resonance could easily be excited by this wave.

As the authors have demonstrated convincingly, the damping factor $2\beta\omega$ of Eq. (19) is very small and for most practical purposes is negligible. This was stressed, too, by Abramson and the writer (1955) and has been remarked upon by Russell (1957) and by Joosting (1957). Eq. (19) for n = 1 thus reduces to

$$\ddot{x} + \omega^2 x = V\sigma \sin \sigma t \tag{31}$$

and would define the non-resonant, in-phase simple harmonic motion directly correlated with the surface slope as found by the authors.

Eq. (31) can also be written

$$m\ddot{x} + \frac{R}{C_M} = mg\ KA\ \sin \sigma t \tag{32}$$

where R is the restoring force. Since the quantity representing the excitation on the right hand side of Eq. (32) was computed by the authors as -mgs, where s is the water surface slope, and the restoring force calculated as

($-mgs - m\ddot{x}$), it follows from Eq. (32) that this calculated force, R_C, is really

$$R_c = \frac{R}{C_M} \tag{33}$$

and that therefore the true or measured restoring force should be $R = C_M R_C$. This implies that the measured forces should have been up to some 20% larger than those calculated (for C_M from 1.0 to 1.2), whereas the authors have found almost the contrary in Fig. 13. The writer is unable to account for this rather considerable discrepancy other than to suggest that the estimation of the ship movement from the forces in the moorings may have been unreliable owing to the inevitable hysteresis effect that is characteristic of mooring line loading. The possibility that a sign error in the sum ($-mgs - m\ddot{x}$) may have led to the discrepancy is ruled out since, for in-phase response, the excitation ($-mgs$) and the ship acceleration ($\ddot{x}$) are always of opposite sign, leading to a simple arithmetical sum of the two terms, as taken by the authors. The authors' results are otherwise a very nice demonstration of the expected linear dependence of the ship movement and restoring force upon the water surface slope.

REFERENCES

1. Abramson, H. N. and Wilson, B. W. (1955); A Further Analysis of the Longitudinal Response of Moored Vessels to Sea Oscillations, Proc. Joint Mid-West Conf. Solid and Fluid Mechanics, Purdue Univ., Purdue, Sept. 1955, pp. 236-251.

2. Deacon, G. E. R., Russell, R. C. H., and Palmer, J. E. G. (1957); Origin and Effects of Long Period Waves in Ports, Section II, Communication 1, XIXth International Navigation Congress., London, July 1957, pp. 75-93.

3. Joosting, W. C. Q. (1957); Investigation into Long Period Waves in Ports, Section II, Communication 1, XIXth International Navigation Congress, London, July 1957, pp. 205-227.

4. Lamb, H. (1932 Edn.); Hydrodynamics, Cambridge Univ. Press, England, 1932.

5. Wendel, K. (1956); Hydrodynamic Masses and Hydrodynamic Moments of Inertia, Translation 260, David Taylor Model Basin, Navy Dept., 1956, 80 pp.

6. Wilson, B. W. (1951); Ship Response to Range Action in Harbor Basins, Trans. ASCE v. 116, 1951, pp. 1129-1157.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY 7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

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AUGUST: 1725(HY4), 1726(HY4), 1727(SM3), 1728(SM3), 1729(SM3), 1730(SM3), 1731(SM3), 1732(SM3), 1733(PO4), 1734(PO4), 1735(PO4), 1736(PO4), 1737(PO4), 1738(PO4), 1739(PO4), 1740(PO4), 1741(PO4), 1742(PO4), 1743(PO4), 1744(PO4), 1745(PO4), 1746(PO4), 1747(PO4), 1748(PO4), 1749(PO4).

SEPTEMBER: 1750(IR3), 1751(IR3), 1752(IR3), 1753(IR3), 1754(IR3), 1755(ST5), 1756(ST5), 1757(ST5), 1758(ST5), 1759(ST5), 1760(ST5), 1761(ST5), 1762(ST5), 1763(ST5), 1764(ST5), 1765(WW4), 1766(WW4), 1767(WW4), 1768(WW4), 1769(WW4), 1770(WW4), 1771(WW4), 1772(WW4), 1773(WW4), 1774(IR3), 1775(IR3), 1776(SA5), 1777(SA5), 1778(SA5), 1779(SA5), 1780(SA5), 1781(WW4), 1782(SA5), 1783(SA5), 1784(IR3)[c], 1785(WW4)[c], 1786(SA5)[c], 1787(ST5)[c], 1788(IR3), 1789(WW4).

OCTOBER: 1790(EM4), 1791(EM4), 1792(EM4), 1793(EM4), 1794(EM4), 1795(HW3), 1796(HW3), 1797(HW3), 1798(HW3), 1799(HW3), 1800(HW3), 1801(HW3), 1802(HW3), 1803(HW3), 1804(HW3), 1805(HW3), 1806(HY5), 1807(HY5), 1808(HY5), 1809(HY5), 1810(HY5), 1811(HY5), 1812(SM4), 1813(SM4), 1814(ST6), 1815(ST6), 1816(ST6), 1817(ST6), 1818(ST6), 1819(ST6), 1820(ST6), 1821(ST6), 1822(EM4), 1823(PO5), 1824(SM4), 1825(SM4), 1826(SM4), 1827(ST6)[c], 1828(SM4)[c], 1829(HW3)[c], 1830(PO5)[c], 1831(EM4)[c], 1832(HY5)[c]

NOVEMBER: 1833(HY6), 1834(HY6), 1835(SA6), 1836(ST7), 1837(ST7), 1838(ST7), 1839(ST7), 1840(ST7), 1841(ST7), 1842(SU3), 1843(SU3), 1844(SU3), 1845(SU3), 1846(SU3), 1847(SA6), 1848(SA6), 1849(SA6), 1850(SA6), 1851(SA6), 1852(SA6), 1853(SA6), 1854(ST7), 1855(SA6)[c], 1856(HY6)[c], 1857(ST7)[c], 1858(SU3)[c].

DECEMBER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(SM5), 1864(SM5), 1865(ST8), 1866(ST8), 1867(ST8), 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876(WW5), 1877(CP2), 1878(ST8), 1879(ST8), 1880(HY7)[c], 1881(SM5)[c], 1882(ST8)[c], 1883(PP1)[c], 1884(WW5)[c], 1885(CP2)[c], 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(HY7), 1891(PP1).

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c. Discussion of several papers, grouped by divisions.

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